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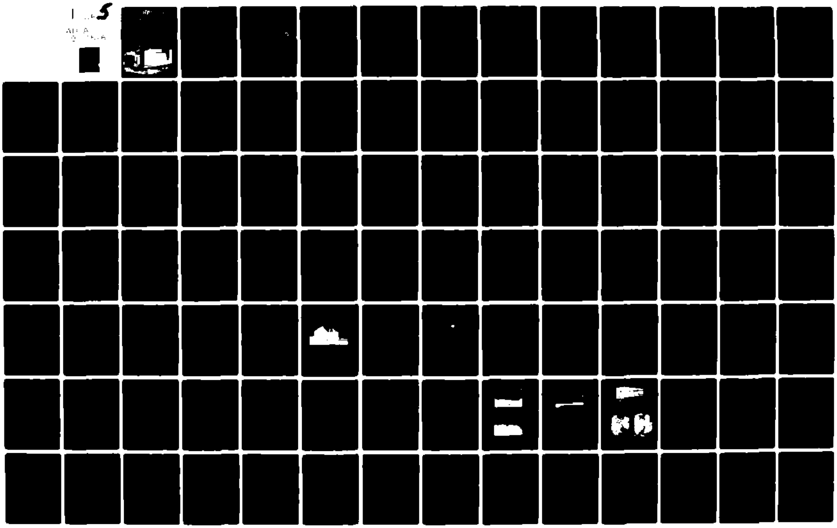

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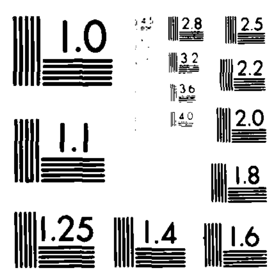

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U.S. Department of Housing and Urban Development

U.S. Army Corps of Engineers

LEVEL II

Building under Cold Climates and on Permafrost

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Collection of Papers from a U.S.-Soviet Joint Seminar, Leningrad, U.S.S.R.

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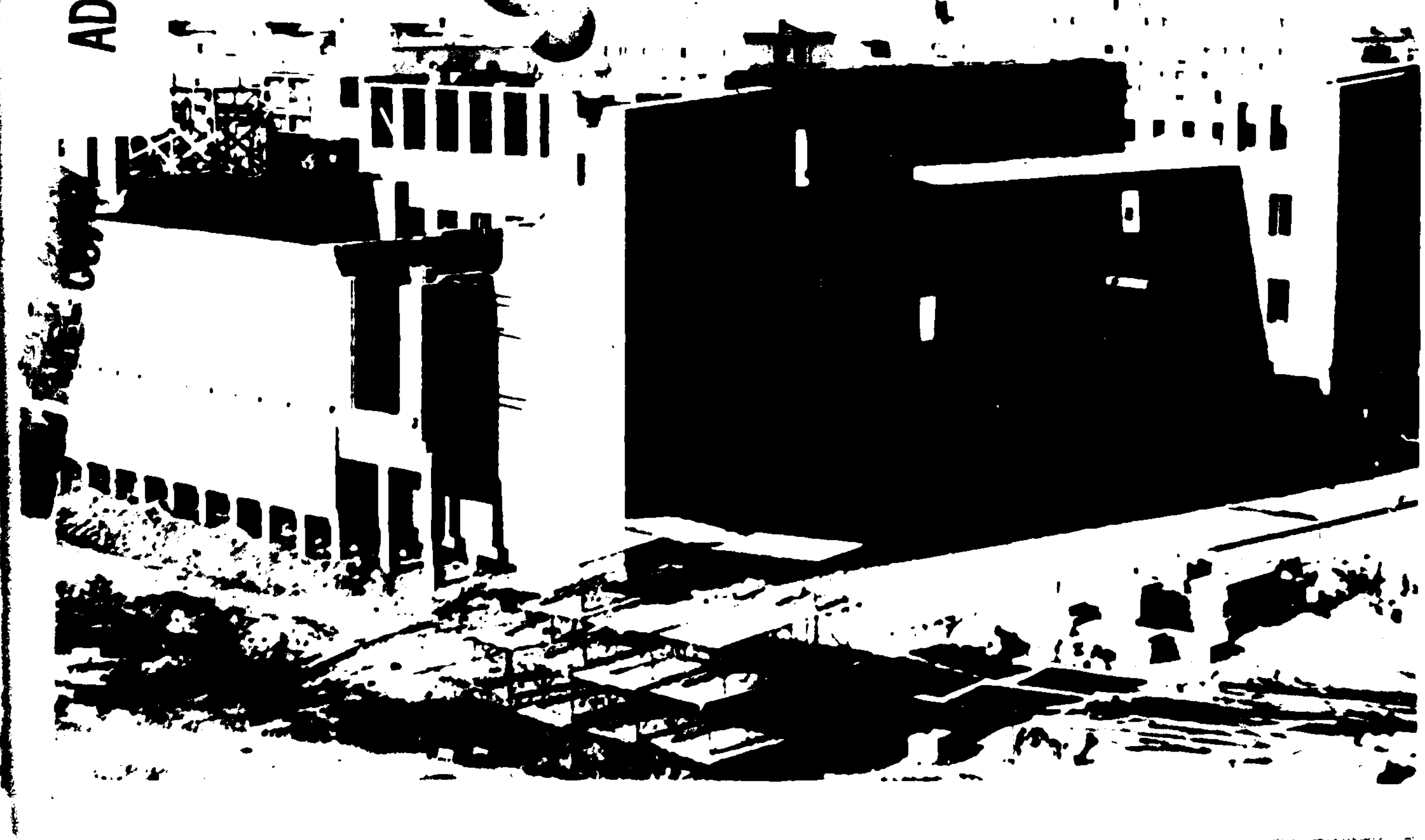

Cover: Masonry building, elevated above permafrost on piles, under construction in Yakutsk, U.S.S.R., 1973. (Photograph by W. Tobiasson.)

BUILDING UNDER COLD CLIMATES AND ON PERMAFROST

COLLECTION OF PAPERS FROM A US-SOVIET JOINT SEMINAR, LENINGRAD USSR

U.S. DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
Office of International Affairs

U.S. ARMY CORPS OF ENGINEERS
Research and Development Office

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~~December 1980~~

PREFACE

The building of homes and other structures in cold weather poses special design and logistical problems for architects, urban planners, and construction engineers. As the United States expands development in the Arctic and Subarctic regions of North America, access to the research and achievements of other nations experienced in cold weather construction becomes increasingly important. The Soviet Union, with so much of its vast territory lying in the far north, performs about 85 percent of the world's research in this field. For this reason, experts at the U.S. Army Corps of Engineers have actively cooperated with Soviet experts under the framework of the U.S.-U.S.S.R. Agreement on Cooperation in the Field of Housing and Other Construction. This publication is a product of their joint activity thus far.

The U.S. Department of Housing and Urban Development, in conjunction with the Corps of Engineers, is very pleased to publish this collection of papers reporting on some of the most advanced Soviet and American techniques for building under extreme weather conditions. It is intended to assist American builders and planners involved in Corps- and HUD-sponsored projects in cold climates. We also hope it will reach a broader audience of professionals in the urban planning and construction fields, both here and abroad, who should find it useful as they carry out further research and apply new technology in solving specific cold weather construction problems.

Tila Maria de Hancock
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for International Affairs
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U.S.-U.S.S.R. Housing Agreement

FOREWORD

The papers in this collection were originally prepared for a seminar held in Leningrad in June 1979 entitled "Building Under Cold Climates and On Permafrost." The seminar was a joint effort conducted under the U.S.-U.S.S.R. Agreement on Cooperation in the Field of Housing and Other Construction, which was signed in 1974 by the President of the United States and the Chairman of the U.S.S.R. Council of Ministers. The U.S. Department of Housing and Urban Development (HUD) and the U.S.S.R. State Committee for Construction Affairs (GOSSTROI) were designated as the Executive Agencies to coordinate Agreement activities. Other government agencies in both countries also participate in this bilateral program.

One of the six Working Groups established to implement technical information exchanges under the Agreement is "Building for Extreme Climates and Unusual Geological Conditions," referred to as Working Group 10.05. The U.S. Army Corps of Engineers and GOSSTROI, lead agencies for this Working Group, co-sponsored the June 1979 seminar.

The seminar brought together Soviet and American architects, planners, builders, environmental scientists, and engineers in an attempt to exchange information on topics that included regional planning and architectural development in cold regions, environmental considerations in northern communities, and innovative building techniques. Of particular importance were presentations addressing building foundations for seasonal frost and permafrost conditions. Among the 21 papers presented by experts from both countries were state-of-the-art surveys as well as reports on new technological developments in cold weather construction.

The U.S. seminar co-chairmen gratefully acknowledge the contributions of each participant and author, especially Andrew Assur for his continuing efforts to improve international technical cooperation. Special thanks are also extended to Nancy Cummings who arranged for the translation of the Soviet papers; George K. Swinzow who edited the English translations; David Moon who was responsible for final editing and format design of the collection; and the staff of the Word Processing Center of the U.S. Army Cold Regions Research and Engineering Laboratory, who provided extensive assistance in preparing this volume.

Finally, publication of these papers would not have been possible without funding from the U.S. Department of Housing and Urban Development, Office of International Affairs, whose support is greatly appreciated.

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U.S. Co-Chairmen of the Seminar

INTRODUCTION

The Working Group on Building for Extreme Climates and Unusual Geological Conditions (10.05) is co-chaired by the Chief of Research and Development, U.S. Army Corps of Engineers, and a Deputy Chairman of the U.S.S.R. State Committee for Construction Affairs (GOSSTROI). Scientifc and technical cooperation is carried out in four subject areas: cold regions engineering; foundation design for unusual geological conditions (except frozen ground); construction and construction management under difficult climatic conditions; and design and construction of reservoirs, smokestacks and cooling towers.

Among the recent achievements of Working Group 10.05 are long-term exchanges of experts to study design and construction technology; shorter-term visits to observe and discuss on-going research at various scientific and academic institutions; and informal workshops on specific topics. The seminar in Leningrad was perhaps the most significant achievement. Not only did it involve assembling some of the world's foremost cold regions experts, but also required tedious and persuasive negotiation to mold the concept into a reality.

One of the long-range goals established by Working Group 10.05 is the eventual publication of joint monographs to be used by engineers and technologists working in construction, design, and R&D organizations directed toward solving design and construction problems in cold regions of the United States and the Soviet Union. Cooperative work leading toward that goal includes the dissemination of information on specific topics -- the design and construction of foundations on permafrost, construction methodology for low temperatures, equipment used for working frozen or permafrozen soils -- collected through different exchange activities. Having these objectives, the Working Group agreed in December 1977 to organize its first joint seminar. Both sides also agreed to publish, in their respective languages, the information presented and discussed at the seminar.

The late Professor G.V. Porkhaev, Deputy Director of the Research Institute for Foundations and Underground Structures (NIIOSP) in Moscow, and Mr. V.V. Sudakov, Director of the Leningrad Zonal Research and Design Institute for Residential and Public Buildings (LenZNIIEP) were designated as the Soviet co-chairmen responsible for planning and coordinating this seminar on the U.S.S.R. side. Mr. A.V. Sadovsky, Deputy Director, NIIOSP, succeeded Professor Porkhaev. Mr. E.F. Lobacz, Chief, Civil Engineering Research Branch, U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, and Mr. Harlan E. Moore, Chief, Foundations and Materials Branch, U.S. Army Engineer District, Alaska, Anchorage, were assigned corresponding responsibilities on the U.S. side.

The seminar was held June 24-29 1979 at the historical House of Architects in Leningrad, and was hosted by LenZNIIEP. Thirteen American and eight Soviet papers were prepared for the seminar. However, not all authors were able to attend. It is worthwhile to note the broad spectrum of contributing authors. From the U.S. they included private consultants,

educators, professional builders and researchers. The Soviet authors included educators and research personnel from the government agencies dealing with building, and architectural and engineering research and administrative organizations. Most authors discussed the applications of new techniques and performance evaluation from a miltidisciplinary approach, oriented toward solving the problems of general construction engineering in cold climates and on permafrost.

The articles in this collection have been classified into the following five sections:

1. Aspects of Architectural Planning, Construction and Environmental Considerations;
2. Principles of Foundation Design and Behavior;
3. Foundation Stabilization;
4. Concrete Construction; and
5. Excavation Techniques.

It is believed that this arrangement will readily simulate the order of considerations typically involved in general construction projects; that is, the planning, design and construction sequence. It should be noted that editing of the manuscripts has been kept to a minimum and was intended only to maintain an essential consistency of style.

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3. H. Moore presented "Thermal Behavior in a Large Concrete Structure in Interior Alaska" by F.A. Anderson.

4. E.F. Lobacz presented "Using Exposives to Excavate Frozen Ground" by R.G. Tart and L.L. Oriard.

5. G.C. Hoff presented "Regulated-Set Concrete for Cold Weather Construction" by F.H. Sayles and B.J. Houston.

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NIIOSP - Research Institute of Bases and Underground Structures, Moscow

LenZNIIEP - Leningrad Zonal Scientific Research and Design Institute for Residential and Public Buildings, Gosgrazhdanstroy

WNIIST - Ministry of Construction for Oil and Gas Industry, Moscow

TSNIIOMTP - The Central Research Institute for Foundations and Underground Structures, Moscow

GOSSTROI - The State Building Committee of the USSR Council of Ministers, Moscow

FUNDAMENT PROJECT - Government Design Institute for Bases and Foundations, Moscow

AMDERMA LABRATORY on the shore of the Kara Sea, 230 km west of the Urals, in the European Arctic

SECTION I:

ASPECTS OF ARCHITECTURAL PLANNING, SYSTEMS ORGANIZATION, BUILDING CONSTRUCTION AND ENVIRONMENTAL CONSIDERATIONS IN COLD REGIONS DEVELOPMENT

ARCHITECTURAL PLANNING SOLUTIONS OF URBAN COMPLEXES: STANDARD RESIDENTIAL AND PUBLIC COMPLEXES IN THE FAR NORTH

By G.D. Platonov[1]

INTRODUCTION TO PLANNING SYSTEMS

The Far North of the USSR, occupying almost half of the entire country, is approximately equal in size to the territory of Canada, Alaska, and Greenland taken together. Because of the active industrial utilization of this region in the last 35-40 years its population exceeds the population of the foreign North by 15-20 times. More than 40 cities and 400 towns are located here. The population of the largest city of the Far North - Murmansk - is more than 300,000 and four cities, Noril'sk, Vorkuta, Yakutsk, and Magadan, have populations of over 100,000 people.

The development and creation of large industrial centers for the extraction and processing of mineral resources in the Far North has required a significant amount of residential and public construction. In recent years, up to 100,000 well built apartments, hundreds of school, kindergarten and day-care buildings, clubs, stores, restaurants, and so forth have been built here.

The development and construction of cities and towns is taking place in practically virgin territory; therefore, planning documentation is preceded by scientific research in determining the most efficient system of settlement for a given region. The development of industry, electric power systems, construction supply bases, the required level of social services, and so forth are considered here.

In practice, two systems of settlement are used in the Far North, group and duty-expeditionary. The group system of settlement involves the creation and development of a group or the conglomeration of permanent cities and towns (sometimes even connected by their industrial nature), connected by one system of external transportation, social services, and a base for the construction industry.

The duty system involves the creation of new, or the use of existing permanently settled points - base cities with the development of a social-economic infrastructure for servicing the workers and their families. Duty settlements for the temporary location of personnel on duty are created at the work sites, located beyond the limits of daily accessibility. These settlements are residential complexes of the hotel type with elements of social services designed for satisfying the everyday primary requirements of on-duty personnel, working without leave for 7-15 days. All periodic and episodic services are performed in the base city.

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For the Far North, as for all regions of the USSR, there is a single system of development of project planning documentation obligatory for all ministries and administrations and confirmed by the State Committee of the Council of Ministers of the USSR for Construction Affairs (Gosstroy SSSR).

The construction standards and rules regulate the number of places in each of the service institutions per thousand inhabitants, figuring on the established area of 12-15 m^2 per person. In order to create personnel and acclimatization of the population arriving in the Far North, the administration envisages not only a higher level of pay, Polar bonuses, and paid leave after 42 working days, but also a higher level of comfort in residential and public buildings.

The existing system of planning for the construction of cities and towns in the USSR is aimed at solving the most important State task - the creation of the necessary conditions for the formation of a permanent population in the Far North.

DETERMINANTS AND CLASSIFICATION OF DEVELOPMENT PLANS

A characteristic feature of Soviet architecture is the municipal planning foundation for all of its solutions. In specific planning systems for towns, planners base their work on providing the population with the best working, living, and recreational conditions with the most efficient utilization of the territory.

Municipal planning principles and procedures, aimed at the creation of comfortable living conditions for the population in a severe climate, are being perfected in the Far North along with industrial and economic development.

Good organization of the residential environment of a northern town is provided by the neutralization of the influence of unfavorable natural climatic conditions, and also by maximum consideration of the social requirements of the population.

The basic structural unit of a settlement is the residential complex - a close, compact residential and social service unit, designed as a number of relatively closed and protected spaces grouped around an axial connection with general urban social zones. Small and average sized settlements are created on the basis of the same principles.

Urban planning and development in the North are determined by the following factors:

The territorial isolation of groups of inhabited points and industrial installations

The small number of inhabitants of most settlements

The greater distance between inhabited points as compared with the central zone of the country

The one-sided nature of the industry, basically the mining industry

The poor ground transportation links with developed regions of the country.

The severe climatic conditions, in particular, low temperature, strong winds and blizzards, require a special approach to the development of a residential complex. Therefore, it is necessary to carry out comprehensive scientific investigations aimed at determining the laws governing the interaction of environmental factors with residential and public buildings. This research is carried out with the use of modeling and special equipment or in full-scale conditions.

Models both of individual buildings and of whole blocks were created for studying the influence of a group of buildings on the snow and wind flow. The models were tested for wind resistance in a wind tunnel, and for snow resistance in a blizzard tunnel.

The nature of the snow accumulation was determined on models of buildings which were subject to the influence of external meteorological factors for a long period of time. In addition, actual measurements of wind speed and direction, solar radiation, the rate of snow drifting during blizzards, and so forth were carried out for a complex investigation of extreme conditions. Definite laws governing the processes by which the drifting and accumulation of snow in a housing development take place were determined as a result of all of this. Recommendations for efficient planning procedures were developed on the basis of these investigations: 1.) the form and parameters of the development, 2.) the types of buildings (wind resistant houses), 3.) structural elements, and so forth. The above mentioned scientific developments were used in construction-climatic zoning of the North, and the construction planning regionalization of both the entire northern territory, and of indvidual regions (Yakutsk ASSR and Magadan region). An analysis of the natural climatic, socio-economic, and construction characteristics was performed in order to provide the basis for a division of the territory into regions. Construction planning regions were defined as territories with unique fundamental requirements for housing development depending on natural climatic conditions, and as territories within the sphere of influence of long term housing construction units. It is evident that it is necessary to use a unique series of standard projects within the limits of construction planning regions.

All small architectural forms and different open areas in a development are a supplement to the planning solution; they improve the micro-climate of the territory in a complex. Considering the significant number of days with snow storms and blizzards, playgrounds and other types of open areas are covered or placed on supports to reduce their snow load.

The following municipal planning classification of inhabited points was proposed with regard to the future:

Type 1. Support cities, belonging to the "large" class, with a future population of 100-300 thousand inhabitants (Yakutsk, Magadan, and

others). They are being developed on the basis of capital multistoried buildings.

Type 2. Base cities and towns - centers of industrial regions and units with populations from 15 to 60,000 inhabitants. It is recommended that they be developed on the basis of capital, basically 4 and 5 story buildings.

Type 3. Single profile inhabited points - central settlements for placer mines and with other city forming functions - population from 2 to 6,000. They may be formed either on the basis of capital 4 and 5 story buildings, or single story buildings of prefabricated construction, depending on their location in the sphere of influence of a future housing construction unit or in particularly isolated regions.

Type 4. Rural settlements - central settlements of kolkhozes and sovkhozes with from 0.6 to 2.5 thousand inhabitants and settlements of agricultural sections with from 0.5 to 0.7 thousand inhabitants. They are formed on the basis of wooden buildings of few stories or of capital 4 to 5 story buildings.

Type 5. Mobile settlements of construction crews, geologists, hunters and deer herders with different numbers of inhabitants. They are formed on the basis of mobile or prefabricated homes.

Relying on the results of investigations of the natural climatic conditions of the North, experimental survey planning, making it possible to determine the characteristic procedures for the planning and organization of a settlement territory for different climatic regions of the North, is performed. Many of these proposals have been implemented in the construction of Noril'sk, Nadym, and other cities (wind protection groups), and in the plans of Udachnaya and an experimental micro-region in Yakutsk (cryptoclimatic groups).

The investigation of the characteristics of the planning organization of several points in the North also made it possible to formulate a number of municipal planning requirements for designing residential and public buildings.

BULK PLANNING SOLUTIONS FOR RESIDENTIAL HOUSES

The continuous perfection of methods of standardization, the development of standard plans for buildings, and the industrialization of construction work are the general line of development of all mass residential construction. The development of residential civil construction in the USSR takes place on the basis of State plans for social and economic development, correlated with planning for an improvement in the welfare of the people.

The basic parameters of the residences in the USSR are regulated by construction standards and rules obligatory for all planning and construction organizations, which are periodically revised every 7-10 years.

Residential buildings are a basic part of the development of inhabited points and by their parameters they determine the scale of construction. Basically, they create the bulk spatial appearance of the centers of cities and towns, main roads and the internal part of inhabited territory.

In recent years, standard 4 to 9 story industrially manufactured buildings have developed, taking account of the special requirements for internal planning and orientation, and have become common in the large cities of the North. This makes it possible to increase the density of development, to create wind and snow protection for inhabited territory and to differentiate buildings, taking account of the demographic structure of the population. It should be noted here that the height of a story in residential houses for the North is 3 meters (in the middle latitudes - 2.7 meters).

Special requirements are imposed on the design of the first stages of residential buildings, which, as a rule, are used as non-residential areas (commercial floors, passage ways along buildings for communication with public institutions), because of the specific ground conditions (permafrost). A number of areas for collective use are envisioned here: carriage and sleigh rooms, rooms for storing out-of-season things, and in individual cases recreation rooms for children and adults.

The block section method is the basic method for standard planning of a northern residence, as well as for the country as a whole. This makes it possible to satisfy different municipal planning requirements, creating different development procedures. Six block section houses are provided for in the nomenclature of standard residential buildings developed by the LenZNIIEP and recently confirmed by the Gosgrazhdanstroy.

The great number of small family houses in northern cities requires the planning of the appropriate block sections or special houses, and also the appropriate forms of social services in the development

Experience in residential construction in the North has shown that the necessity for compactness of bulk planning designs for buildings is connected with the simplification of the configuration in the plane. Rejection of facade plastics and a drop in heights significantly limits the possibilities of providing for artistic expression in architectural designs of residential houses and residential development as a whole.

The elements of the facade have important significance in the creation of the architectural character of the northern building; these are: windows, alcoves, loggias with transforming glass work, wind protection ribs, entrances, heated outer vestibules, and so forth. These structures, fulfilling the role of local regulators of the micro-climate, simultaneously are architecturally decorative elements of the community.

In choosing color compositions the psycho-physiological and physico-thermal significance of a color is considered.

Most standard plans for residential buildings are designed for complete prefabricated construction and industrial production at house con-

struction plants. The different regional series of residential buildings depend, basically, on the local conditions, the raw material base, and those construction plants which operate in the regions.

Moving to the characteristics of the organization of the internal space of a residential building, in particular a living compartment designed for occupation by families of different demographic composition (from 1-6 persons), it is necessary to note that ten types of apartments having from 1 to 5 rooms are used. The functional organization of northern living compartments forms a certain type of building with respect to the orientation and number of stories, in accordance with the arrangement of utility areas of the apartments along the front of the building.

Architectural planning solutions follow the principles of zoning areas into daytime and night-time zones, protecting living areas from the action of the prevailing winds, a comfortable design of the entry part of the apartments being a well developed set of built-in closets (drying chambers, pantries, closets for storing clothes and shoes). The kitchen-dining room zone is organized taking account of the great air-tightness of northern apartments and the preferred preparation of food on electric stoves, which makes it possible to obtain greater flexibility in the joint use of a kitchen and kitchen-dining room with a common room. According to the standards set, northern apartments have 10% more living space than apartments in the middle latitudes.

Continuous scientific planning for a future more comfortable northern residence in a functional and technical regard, and with respect to optimization of communication with the environment, is being carried out for the next stage of construction (beyond the limits of 1980). Therefore, specific sociological observations are being carried out in experimental residential buildings, after which the program for standard planning and construction is being corrected. The inner space of the buildings includes not only living compartments but also a comparatively limited set of public service elements. Such experimental residential "new type" homes are planned for Vorkuta and Noril'sk. They are already quite modern and can provide for a saving in housework time of 1.3-1.5 times the average because of the closeness of domestic and cultural areas to living areas. In addition, there is an advantage in the architectural composition plan thanks to the larger bulk spatial constructions.

Scientific planning for even more perfect types of dwellings, the so-called "house complexes" with a highly developed structure of service areas, including multifunctional halls, gymnasiums, winter gardens, and so forth, are being developed for the distant future (at the level of the year 2000). Closeness of services to residences will lead to a general increase in comfort which has a very great significance to people living in extreme conditions.

CLASSIFICATION AND PLANNING SOLUTIONS FOR PUBLIC BUILDINGS

These buildings, together with residential homes, make up the standard architectural base of the towns, residential regions and micro-regions.

The planning and construction of public buildings, forming the development of a micro-region or town, also is carried out on the basis of standards which are regulated by State construction departments. These standards take account of the diversity in the natural climatic regions of the country including the differences between individual regions of the Far North. The standards are differentiated depending on the size of the inhabited point. Service radii are provided (the distance from a residence to the appropriate service institution).

The construction of large scale types of public buildings in northern regions, as in the country as a whole, is carried out according to standard plans. The nomenclature for these plans was created by LenZNIIEP and confirmed by Gosgrazhdanstroy. It includes 82 types of public buildings and structures for different purposes: instructional and commercial, medical, sanitariums, resorts, hotels, sports, theaters, administrative, and so forth.

In order to determine the value of the development of a standard design, the expected frequency of utilization is determined in relation to the regional conditions, and also to the presence of a construction base, the need for a certain building to be connected with specific socio-demographic factors, and so forth. No less than 10 times the expected frequency of utilization of a given building over the course of a five year period serves as a criterion for determining the necessity for the inclusion of a given building in the nomenclature.

The standard planning of public buildings for the Far North is concentrated in the zonal institutes of Gosgrazhdanstroy (LenZNIIEP SibZNIIEP). The remaining standards of plans are developed by a specialized institute (for example, Giprozdrav, the Ministry of Health of the USSR, and others). This system provides for a high level of functional, technological, and structural solutions as a result of the concentration of experienced architects, builders, and technicians in the large planning organizations. Here it is possible to create scientific subdivisions, experimental bases, and peripheral departments which are capable of furthering projects and monitoring their utilization.

The creative exploration of functional planning, compositional and artistic solutions is accompanied by the thorough construction plans and the utilization of new, more efficient construction materials and systems of engineering equipment. The bulk spatial construction of all public buildings, in connection with extreme climatic conditions, is based on the idea of maximum compactness, a consequence of which is the greater width of the building, the absence of drops both in height and along the horizontal of the building, and sometimes streamlining. The architectural composition is determined by the lapidary forms; therefore it is necessary to use elements incorporating art, and, in the first place, the active introduction of color. A public building should stand out clearly against the background of snow covered spaces and the general residential development.

Preschool institutions for children, combined day care centers and kindergartens, which perform a particularly important social function (freeing women for socially useful work), are the basic, most mass produced type of public building in our country. The most common capacity for cities

of the Far North is 280 places, and 90 and 140 places for villages.

Because of the peculiarities of the demographic structure of northern inhabited points (with the presence of small villages) it becomes necessary to develop a cooperative type of children's institution and primary school. In comparison with the middle belt, the northern day care center/kindergartens are distinguished by greater comfort as a result of the limited time the children can spend in the open air.

General education schools are the second most common type of public building. Universal average education has been introduced in the USSR, in accordance with which the list of types of buildings has been determined. Plans for 20 classes (784 students) and for 30 classes (1176 students) are used for city development. For settlements of the urban type and rural inhabited areas, schools of lower capacity are used: eight year schools for 192 and 320 and ten year schools for 392 students. In comparison with schools designed for the middle belt, buildings for the Far North have larger areas set apart for entrance halls, closets, and recreational areas; a nature lecture and display room and a photarium have been introduced. A number of experimental designs also have included swimming pools and winter gardens. In recent years compact wide school buildings have been built in cities in the northern zone.

Commercial centers for micro-regions, units offering primary services for a group of residential houses, and general village centers are the third common type of public building. Standard designs for public buildings for micro-regions of 6 and 9 thousand inhabitants have been developed in LenZNIIEP and SibZNIIEP. Institutions of everyday use are combined in these buildings: commercial, public feeding, domestic services. Primary service units, including approximately the same structure of institutions, are designed for 2 to 3 thousand inhabitants. Village complexes (for 0.5 and 1.0 thousand inhabitants) are designed in the form of a single building; the centers of larger villages consist of several interconnected units, connected directly with one another or with the use of heated passages. These buildings are from 1 to 3 stories high.

The practice of recent years has confirmed the expediency of using the block structure for building centers which provides for great flexibility in municipal planning and the introduction of individual buildings in stages. General villages of inhabited areas with 9 to 12 thousand inhabitants or more are constructed according to individual plans (since the frequency of building them is very small).

CONCLUSIONS

Proposals for the construction of administrative complexes in cities and villages (the regional centers of the North) are being developed on the same principle of cooperation and blocking. These combine local governments, social organizations, financial institutions, courts, police, a number of small economic administration units, communication institutions, computer centers, and so forth.

Specialized public buildings in small, medium, and large cities such as

clubs, theaters, gymnasiums, swimming pools, hotels, and medical institutions are also built according to standard plans, in which the specific features of the North are considered both in the planning organization and in special construction solutions.

Experimental planning of different types of public buildings is carried out for the purpose of perfecting the functional planning structure and the architectural and artistic composition for the next stage of construction. Thus, for example, day care-kindergartens are designed with health improvements in mind: their internal structure includes swimming pools, verandas, and winter gardens. Similar improvements are planned for school buildings. The planning experiment also encompasses larger sports institutions: swimming pools, covered skating rinks, and lighted ski runs. Putting these into operation will make it possible to overcome the consequences of hypodynamia, which is especially dangerous in northern conditions where people have sharply limited mobility, by architectural and municipal planning means.

Completely new types of public sports buildings such as "cross pavilions" and "beach-swimming pools" (they already exist in scientific hypotheses and speculative plans) may be used in the more remote future.

THE USE OF SYSTEMS THEORY IN THE ORGANIZATION OF CONSTRUCTION MANAGEMENT IN THE FAR NORTH

By B.J. Berezovsky[1]

INTRODUCTION

The problem of settling new regions by means of the uniform resettling of different groups of populations for the settled form of life has been solved in the conditions of the Middle Belt of our country. For this purpose a complex industry providing for the occupation of all members of the family has been created. Resettling in the conditions of the Middle Belt assumes the development of regional and interregional connections, and also the gradual development and growth of the populated point.

In the conditions of the Far North, resettlement is entirely subject to the problems of the development of natural resources located in unfavorable climatic conditions. The characteristic development of a narrowly specialized industry creates an acute occupational problem for secondary members of the family. The inaccessibility of central industrial complexes does not lead to the development of regional connections (although the latter are a necessary condition for central settlements in the Far North).

In order to study the entire diversity of the interacting factors it is necessary to have a satisfactory model which will be a representation of the dynamics of the interconnected processes influencing construction. This problem should reflect the interconnected factors, representing a single chain, taking account of feedback.

The theoretical principles for the creation of complex models, and the methods of investigating them, are the product of a developing branch of science - the general theory of systems.

The development of the North may be represented in the form of the complex goal-directed system, including a determined system of construction, leading to the solution of general problems of development.

QUALITATIVE SYSTEMS MODELING

The methodological basis (core) of the system of construction consists of a number of sequential interconnected questions: <u>why, for whom, what, and how to build</u>?

The methodological basis presented above predetermines the structural model of the problem of construction in the form of a hierarchical multi-level system.

The economic level includes the system of the development of the North (C^o). The disposition of regional systems of economic development of other

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regions of the country (the aggregate of which comprises the total system of the national economy of the country as a whole) is implied on the same level. The system of development may be broken down into subsystems of basic trends in development, which, in turn, may be divided into more detailed subsystems of a regional and industrial nature. The degree of detail depends on the general purposes of the investigation, and should provide for a sufficient volume of information for characterization of the city forming factors.

The most characteristic city-forming factors, for the sake of which construction in the extreme north is performed, are characteristic of the contemporary stage of development, and are named below:

C_1 - support points - ports, located on the shores of the Arctic Ocean for establishing internal transportation links along the northern sea route.

C_2 - extractive industry undertakings based on deposits of the most economically profitable or extremely rare mineral resources.

C_3 - internal transportation-transfer basis for the material and technical provisioning of extractive industrial undertakings.

C_4 - local power generating plants (hydroelectric plants, thermoelectric plants, atomic-electric plants).

C_5 - special locations for servicing communication, aviation, and the fleet; scientific research stations.

The city-forming level also may be subdivided in greater detail. For example, the subsystem of a mining industry establishment may include ore mining, diamond mining, oil and gas producing, coal industry institutions, and so forth. These groups of undertakings, in turn, may be subdivided into smaller systems. For example, the ore mining undertakings may be divided into underground mines and open pit mines.

The mining level makes it possible to reveal individual objects of construction and the parameters for the next level of work resources. On this level, the question "For whom to build?" is investigated, and the amount, structure, and form of the resettlement of labor resources and different groups of population are determined at the stage of geological surveys, construction, and exploitation.

The question "What to build?" is solved at the city construction level and the city construction homes of industrial settlement complexes, the characteristics of the overall planning and structural solution of them, and technological provisions for them are determined.

The information obtained is the source for solving the question "How to build?".

The problems of selecting the type of structural base and providing designs and materials for the construction of types of structures, and also

the technical facilities which make it possible to perform construction assembly operations, are solved on the organization-technical level.

Thus, any level of the system may be subdivided into subsystems of the necessary order, and investigated independently for the purpose of a qualitative and quantitative description of objects and their interconnections.

INTERPRETATION OF ANALYTIC DATA

The results of these investigations may be a number of theoretical models of the structure of industrial operations, the structure of the population, the system of resettling, or the structure of populated places. Three dimensional models of industrially inhabited complexes and service systems (structures of the organization of construction and the production of construction assembly) operations are studied.

In turn, subdivision of the system of the organizational-technical level makes it possible to determine the output of local bases of the construction industry, the necessary volume and type of structural elements (imported from external bases) and also tables of mechanisms required for the performance of planned operations.

The investigation of the organizational technical level should be performed with the use of methods of selecting optimal systems for the organization of construction and assembly operations, consideration of the productivity of the work of people, and mechanisms depending on the time of year, transportation conditions, and the degree of development of the construction site.

Investigation of the state, and in the case of the action of external factors, is performed with the use of auxiliary information systems.

The system of natural climatic factors, developed to the necessary level of detail, may be represented as an information system. Similar systems may be developed for social, economic, psychological and other factors. The system of organization of construction and assembly operations, methods of operating with detail to the level of construction operations, may be presented.

The simultaneous operation of all subsystems in the case of the intentional coordination of these actions is investigated in solving the problems of the control and optimization of the construction process. It should be noted that the construction system is characterized by a certain freedom of choice in the means (paths of action) and the taking of solutions within the limits of the general purposeful setting of the system of development of the North. In connection with this the criteria for evaluating the functioning of the system, as a whole, and at different levels, apply decisive significance.

Scientific prediction involves changing the state of the individual subsystems as a result of the influence of given parameters of scientific-technical and social progress on them. The higher the level of influence is, the more radically does the state of the system as a whole change.

Influence on the highest level may fundamentally transform the structure and purposes of the system. We have in mind such revolutionary shifts as a change in the climate of the extreme north or the depletion of territorial resources. In these cases the directions of development will be determined primarily by social factors and the construction system will change its structure. It is assumed that within the limits of the foreseeable future, it is impossible to count on such radical events. Therefore, containing the overall general system of development as a whole, an examination of certain possible changes in the state of the subsystems (within the limits of several decades) is of practical interest. Of the above city-forming factors, the most stable are evidently transport-support points on the coast of the Arctic Ocean.

With the solution of the possibilities of transAtlantic sailing from the Pacific Ocean to the Atlantic, and with an increase in the possibilities for underwater sailing based on atomic energy, these points will be the connecting link for individual points on the globe with the central regions of Siberia.

Power generating facilities may have a high degree of stability. The possibility of transmitting electric energy significant distances with the use of quantum generators (lasers) will make it possible for power plants to service large territories.

Extractive undertakings are characterized by different degrees of stability and this depends on the following:

1. the periods of exploitation of the deposit (the presence of resources, the capacity of the undertaking, the laboriousness and cost of production)

2. the competitiveness of the resources extracted (rarity, quality, accessibility to transportation).

The development of such undertakings to a significant degree is determined by the level of geological information.

Thus, the tendency for instability of the extractive industry as a city-forming factor will be intensified in the future.

The development of transportation, the perfection of structures and means of mechanization, radically influences the methods of organization of construction and the methods of performing operations. The principles of the organization of industrial construction bases, evidently, are solved at higher levels of the system. It is not the external features of the concentration of amounts of construction or the degree of scattering thereof, but the parameters of city-forming factors, which will be decisive here.

The concepts presented in this report reflect only certain aspects of systematic investigations in the area of construction in the Far North. A qualitative description of the structure, being a necessary initial stage in the scientific investigation of a complex system, can already specify the place and value of individual problems of construction and indicate paths for their solution.

OPTIMUM CONSTRUCTION SYSTEMS

In developing construction systems, the maximum reduction of expenditures of human labor is a basic trend in the technological politics of construction in the North. This can be accomplished by mechanization of design solutions, the elimination of extraneous obstacles in supplying material-technical resources, and the choice of optimum systems of organization of construction, taking comprehensive account of the influence of the complex natural and climatic conditions of the North on production. The achievement of all possible savings in human labor also requires intensification of construction processes and the use of technology in a "northern" variety. Therefore, reducing the time required for the construction of objects, reduction of labor expenditures and costs in performing operations may be performed on the basis of a "transparent" intercorrelation of effective solutions in economic planning, the formation of labor resources, and groups of population, urban construction, organization and production technology.

The selection of optimum urban construction, organization-technical solutions, and construction time must be closely connected with the natural conditions of regions, the size of operations, transportation conditions of material-technical supplies, increased demand for labor force and construction machinery.

In arranging construction bases it is necessary to start from servicing not only individual economic complexes, but industrial units and regions.

In regions with relatively developed internal transportation connections and the presence of a resource serving the economy, it is necessary to create stationary construction bases in the shortest periods of time.

For the development of central economic complexes without the prospect for further development, it is necessary to use mobile bases and (where transportation links are convenient) to deliver structures from the central regions of the country. The delivery of prefabricated collapsible structures from bases located beyond the limits of the northern zone has primary significance in the pioneering development of northern regions.

There are significant reserves for the growth of labor productivity in the area of the technology of construction and assembly operations (in particular in the case of the construction of monolithic structures).

CONCLUSIONS

In the area of the technology of operations, typical solutions and scientific recommendations for more efficient methods and approaches to the performance of operations (taking account of the significant corrections which the climatic and permafrost factors of the north introduce) have been insufficiently developed. This is explained by the complexities of the technology of construction and assembly operations in combination with complex natural factors inherent to the northern climatic zone of the country: the existence of low temperatures down to -50 to -60°C, strong winds to 40 m per second, permafrost, frequent blizzards, polar nights, the absence

of local high quality nonmetallic minerals, difficult transportation conditions, the scattering of construction, difficult accessibility during the spring and summer period (from June to August), and others. Often the impossibility of rapid maneuvering of construction techniques and energy resources is not unimportant.

The completion of a broad program of goal-directed systematic investigations will require the radical re-examination of the organization of scientific research in the area of construction in the Far North.

BUILDING CONSTRUCTION TECHNOLOGY UNDER WINTERTIME CONDITIONS: RECENT EXPERIENCE IN INTERIOR ALASKA

By F. LAWRENCE BENNETT[1]

INTRODUCTION

In his chapter on "The Construction Industry," Professor Zilly states,

> "The Swedes handle winter construction with thermal underwear, the Russians do it with pre-cast concrete and guts, the Canadians do it with modern technology and government support, and the Americans do it reluctantly, if at all. In short, the United States construction industry has been too busy discovering why it cannot build in the winter, not busy enough discovering why it can." (1)

During the past decade, the cold regions of interior Alaska have seen a marked upsurge in construction activity, and, in contrast to Professor Zilly's statement, much construction activity has gone forward during wintertime. The construction of the trans-Alaska oil pipeline, completed in 1977, provided the United States' most striking example of successful construction management in a cold environment. Construction of the pipeline itself, its supporting pump stations, gathering system, communications system, and tidewater port facilities, together with a wide variety of other construction resulting from the pipeline project, made progress under wintertime conditions. Although the pace of construction was slower than in the summertime, progress was made, in contrast to common practice in the past, when most construction work was suspended for several months during the winter.

The purpose of this paper is to describe recent research conducted by the School of Engineering at the University of Alaska which has documented some of the techniques that have been a part of the increased building construction activity in interior Alaska during the wintertime.

After describing the geographical and climatic setting within which construction in the Fairbanks area of interior Alaska takes place, the paper presents the results of a survey of Alaskan constructors which indicates that many types of construction activities can be, and have been, carried out at temperatures well below 0°F (-18°C). The paper then describes some studies dealing with the provision of temporary enclosures and temporary building heating inside buildings under construction in the wintertime. A case study of a building project on the University of Alaska, Fairbanks campus during the 1973/74 winter is discussed, and a computer program that can assist the construction estimator in predicting the cost of providing temporary heating and

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enclosures is described. The section on temporary protection of wintertime building construction concludes with a summary from a survey of several buildings under construction in the Fairbanks area during the 1976/77 winter.

The paper then describes the construction of the roof of an institutional building in downtown Fairbanks, which was carried out during January and February 1977. The roof design is described, the construction schedule and environmental conditions under which the roof was constructed are discussed, and some guidelines for roof construction in the wintertime are suggested.

THE SETTING--INTERIOR ALASKA

The area surrounding Fairbanks, Alaska, is characterized by the broad valley of the middle Tanana River and the surrounding low, rolling uplands. The elevation of downtown Fairbanks and the Fairbanks International Airport is about 435' (133 m) above sea level; the hills surrounding Fairbanks reach elevations between 1,000' and 2,000' (305 m and 610 m) above sea level.

The Fairbanks area is an area of discontinuous permafrost, occurring primarily on the north sides of hills and in the silts of lowland areas, especially where covered with blankets of organic materials.

Interior Alaska is sheltered from maritime influences by mountain ranges on practically all sides. The area has a definite continental climate, with large variations in temperature between winter and summer, in response to large variations in solar heat throughout the year. The average annual temperature recorded at the Fairbanks International Airport is 26°F (-3°C). The average temperature for July, the warmest month, is 61°F (16°C), while the average temperature for January, the coldest month, is -12°F (-24°C). Maximum temperature recorded in Fairbanks was +99°F (37°C) in 1919, while the minimum recorded was -66°F (-54°C) in 1934.

During the months of June and July, the daily average maximum temperatures reach the lower 70's with temperatures of 80°F (27°C) or higher occurring on about ten days each summer. During the period from November to March, extremely cold temperatures of -40°F (-40°C) or colder occur on the average of fourteen days each winter, with extremes of near or below -60°F (-51°C) having occurred in the months of December, January and February. Maximum temperatures during December and January are usually below 0°F (-18°C).

Although snow cover persists during the winter months, snowfalls of 4" (10 cm) or more in a day occur only three times during the average winter; blizzard conditions are almost never experienced.

Ice fog conditions occur frequently at extremely low temperatures, with little wind, and these conditions last for periods of a few days to one or two weeks. Wind speeds are particularly light during the winter months.

The average annual precipitation is about 12" (30 cm), and it follows a fairly regular pattern, with minimum precipitation in April and maximum precipitation in August. (2)

CONSTRUCTION IN THE WINTER--REPORT FROM A SURVEY

To determine the extent to which Alaskan contractors conduct construction operations during the wintertime, the author caried out a survey of contractors doing business in Alaska. (3) The survey was taken in spring 1975, and data were provided by 34 contractors, representing a 40% response of those from whom answers were solicited.

The basic question posed to these contractors was the following:

> "What is the lowest temperature at which the following types of activities can be performed?"

There followed a list of construction activities, such as, "machine excavation," "earth moving and grading," "paving," "roofing," "electrical installation," and "lay out, surveying, etc." Table 1 summarizes contractor response to this question.

A study of Table 1 indicates that responses fell into two categories. Some respondents gave the absolute lowest temperature, while others gave the lowest temperature that would allow efficient operation or the lowest temperature that would be permitted inside an enclosure where the work was being performed. For example, the average response for the question related to machine excavation was -37°F (-38°C) as the absolute low; those reporting on the lowest temperature for "efficient" machine excavation gave an average answer of -1°F (-18°C). Another example is the question on concrete placement. Here, the "absolute low" answer averaged -31°F (-35°C); in this case the respondent is suggesting that the outside temperature can be this low, provided proper enclosures and heating are provided. The average response for the case of concrete placement inside enclosures suggests that the lowest permissible inside temperature is +32°F (0°C).

When asked what factors determine those "cut-off" temperatures below which construction activities must be suspended, the contractors indicated that the effect on people is one of the primary reasons that cold temperatures cause winter shutdowns. Also, equipment is adversely affected by such problems as hydraulic system and metal failure. Further, low temperatures affect such materials as electrical cable (insulation cracking) and soil (freezing). The contractors indicated that, in

Table 1. Summary of Contractor Responses to the Question, "What is the lowest temperature at which the following types of activities can be performed?"

Activity	Absolute Low, °F				Lowest for Efficient Operation or Within Enclosure, °F			
	Responses	Range Lo	Range Hi	Average	Responses	Range Lo	Range Hi	Average
Machine Excavation	15	-70°	-20°	-37°	10	-30°	+32°	-1°
Hand Excavation	11	-70°	+32°	-22°	12	-30°	+33°	+18°
Earth Moving & Grading	11	-45°	-20°	-35°	9	-20°	+32°	+13°
Paving	14	+15°	+45°	+32°	4	+25°	+60°	+41°
Concrete Formwork	11	-50°	-10°	-25°	7	-10°	+33°	+10°
Concrete Placement	6	-50°	-25°	-31°	12	+20°	+40°	+32°
Steel Erection	10	-50°	-20°	-32°	8	-30°	+30°	-1°
Block & Stone Masonry	5	-50°	0°	-25°	12	0°	+40°	+31°
Roofing	7	-30°	+15°	-13°	8	0°	+60°	+24°
Finish Carpentry	5	-50°	-10°	-27°	9	+15°	+50°	+36°
Painting	7	-50°	+40°	+13°	10	+32°	+60°	+42°
Electrical	14	-70°	-10°	-31°	7	+20°	+40°	+28°
Piping & Mechanical	10	-50°	-10°	-27°	5	+20°	+40°	+29°
Pipe Welding	9	-50°	0°	-21°	3	-30°	+33°	+8°
Pre-Built Components	13	-50°	-10°	-28°	6	-30°	+33°	+9°
Layout & Surveying	17	-50°	0°	-33°	4	-30°	+20°	+2°
Loading & Unloading	14	-60°	-10°	-39°	4	-35°	+20°	-1°
Subsurface Exploration	1	-	-	-40°	-	-	-	-

addition to the factor of temperature, other considerations in a decision to suspend wintertime operations include wind, snow, lack of daylight, soil moisture, and icing conditions. They also cited cost of transportation, type of work, size of job, availability of personnel, effect on other phases of the project, union attitude, and quality control as considerations that might keep one job operating and shut another down, even under similar temperature conditions. Money was the other important "other factor" in several responses.

Thus, Alaskan contractors do work in the wintertime. The decision regarding winter work depends on type of operation, temperature, and a host of other factors.

PROTECTION OF BUILDING CONSTRUCTION DURING THE WINTER

The U.S. Army Cold Regions Research and Engineering Laboratory has supported three projects that have been related to the provision of temporary enclosures and temporary heating for buildings under construction during the wintertime. This section describes a "case study" of a typical wintertime project, the development of a computer program for estimating the cost of temporary protection, and a survey of a number of buildings under construction during the wintertime in the Fairbanks area.

I. Laboratory Building Addition, University of Alaska, 1973-74

The Laboratory Building Addition is an extension of the existing Irving Building, located on the West Ridge area of the University of Alaska, Fairbanks campus. (4) The building measures approximately 107' by 70' (32.6 m) by 21.3 m), with three floors in the building, each having a separation of 18' (5.49 m). Construction work began in May 1973 and was completed in late 1974. The contract amount at project completion was approximately $1,785,970, or approximately $79.50 per square foot.

On this project, the contractor used un-reinforced polyethelene sheeting of 6-mil thickness for temporary enclosures for protection from cold weather. The north and east walls on the first floor level were completely enclosed with this material early in the winter season. The structural system consisted of a simple frame of 2" x 4" lumber with the polyethelene sheeting attached to it. In some cases, the sheeting was simply draped, with little wood framing for support. Windows on all three levels were enclosed in a temporary manner, using the same type of polyethelene film attached to wood frames that fitted tightly into the window opening. Temporary entrance doors were fashioned in a similar manner.

Two types of heating devices were used on this project. First, several space heaters were employed, either to provide general area heating, or only to provide contractor personnel with an opportunity to

warm themselves and to dry their gloves and mittens, without providing much warming of the air except in the immediate area near the heater. These heaters, of the "Master" variety, were rated at between 320,000 and 500,000 btu per hour. Consumption of #2 fuel oil varied between 2.6 and 3.5 gallons per hour. These heaters used fans with 115 volt, 20 amp motors.

The other part of the temporary heating system consisted of two steam heaters attached to the University's steam system. Each unit contains a radiator over which a fan blows air; steam passes through the radiator, where it is condensed to water and then returned to the condensate return line. They can be operated either on a manual or an automatic mode, the latter controlled by a thermostat. Each unit is capable of delivering somewhat in excess of 500,000 btu per hour.

An important portion of the Laboratory Building Addition study was the development of an estimate of costs for providing temporary enclosures and heating. The estimated cost of providing materials and labor for enclosures was $4,760, while the cost of equipment, supplies, steam, electric energy and labor for the temporary heating system was $9,350, for a total estimated cost of $14,110. This estimate represents about 0.79% of the constructior contract price. Because only a portion of this building was enclosed and heated, and because construction of that portion contained active operations during less than the full winter period, it was concluded that a reasonable proportion of total contract price to be spent on temporary enclosures and heating for a "typical" project is about 2%.

II. HEATCOST Program

To assist construction contractors in estimating the costs of installing and maintaining temporary enclosures in buildings under construction and providing temporary heating for such buildings, the author has developed a computer programming for estimating these costs. (3) Designated HEATCOST, the program is interactive, in the sense that the user communicates with the computer by means of a remote keyboard terminal in a "conversational" manner responding to various program requests for data and evaluating different enclosure schemes by changing input data.

In response to "prompt" messages from the program, the analyst provides project description; data describing dimensions and heat flow characteristics of each wall, the roof, and the floor; the heating method and the unit cost; expected inside average temperature; monthly maintenance cost percentage; and data on each month during which the system will be in operation, including name of month, number of days, average outside temperature, average soil temperature, and average daily solar radiation.

To illustrate program operation, the terminal keyboard printout for a typical time sharing system is presented as Figure 1. This illustration includes data input for one wall of the building under consideration. Information supplied includes height and width of wall, its heat transmission coefficient, a designator indicating whether it is above or below grade, and a designator indicating which direction it faces (this last designator is used in conjunction with solar radiation).

Input for the wall also includes information on up to three different types of windows. For each type, the analyst supplies the number of windows, their height and width, heat transmission factor, crack infiltration factor, solar radiation transmission ratio, and installation unit costs for labor and material. Similar data are provided for up to three types of doors in each wall. In Figure 1, the computer first prints a "prompt" message, together with an "=" sign, after which the user responds with the requested data.

After all data are supplied, the program totals the estimated heat loss over the life of the project, together with the total estimated cost for installing and maintaining enclosures and providing temporary heating. Figure 2 is a typical output summary, followed by an indication of the manner in which the program can be used to change input data in order to provide a cost estimate for a revised temporary protection system.

To determine how well the HEATCOST program estimates actual heat loss, it was used to calculate heat loss for the University of Alaska Laboratory Building Addition Project, whose actual heat loss during construction had been documented during the 1973/74 winter. (4) Results of this validation run indicate that the program does an acceptable job of estimating heat loss, well within the normal accuracy of building construction estimating.

III. 1976-77 Fairbanks Area Survey

During the 1976-77 winter, the University of Alaska School of Engineering conducted a survey of Fairbanks area construction projects, the purposes of which were to record activities underway at various weather conditions and to document the methods used for temporary protection. Nine building construction projects, with an area totaling over 515,000 sq. ft. (48,000 sq. m), were active in Fairbanks during that winter. They ranged from a shopping mall, containing 130,000 sq. ft. (12,000 sq. m), and a federal office building, with a 154,620 sq. ft. (14,400 sq. m) area to the construction of a series of modular housing units. Three other building projects, which had been started prior to the winter of 1976-77, were not continued through the winter, although their lack of activity was not related to cold weather construction technology. Rather, they suffered either from design delays or financial difficulties.

```
   TYPE WALL DATA FOR WALL  2 --HEIGHT IN FEET,WIDTH IN FEET,U FACTOR,
      BELOW GRADE DESIGNATOR(1. IF BELOW GRADE, 0. IF ABOVE GRADE)
      WALL ORIENTATION DESIGNATOR(1. IF SOUTH FACING,2. IF EAST
      OR WEST FACING, 3. IF NORTH FACING).
=12,80,.67,0,2
   TYPE WINDOW DATA FOR TYPE 1 WINDOWS FOR THIS WALL, IF ANY --
      NUMBER, HEIGHT IN INCHES,WIDTH IN INCHES, U FACTOR,
      CRACK INFILTRATION FACTOR(CF/HR/FT OF PERIMETER),
      SOLAR RADIATION TRANSMISSION RATIO(BETWEEN 0. AND 1.0),
      INSTALLATION LABOR COST/SF,MATERIAL COST/SF.

=1,40,60,.5,250,.1,.65,.02
   TYPE WINDOW DATA FOR TYPE 2 WINDOWS FOR THIS WALL, IF ANY --
      NUMBER, HEIGHT IN INCHES,WIDTH IN INCHES, U FACTOR,
      CRACK INFILTRATION FACTOR(CF/HR/FT OF PERIMETER),
      SOLAR RADIATION TRANSMISSION RATIO(BETWEEN 0. AND 1.0),
      INSTALLATION LABOR COST/SF,MATERIAL COST/SF.

=1,40,25,.5,250,.1,.65,.02
   TYPE WINDOW DATA FOR TYPE 3 WINDOWS FOR THIS WALL, IF ANY --
      NUMBER, HEIGHT IN INCHES,WIDTH IN INCHES, U FACTOR,
      CRACK INFILTRATION FACTOR(CF/HR/FT OF PERIMETER),
      SOLAR RADIATION TRANSMISSION RATIO(BETWEEN 0. AND 1.0),
      INSTALLATION LABOR COST/SF,MATERIAL COST/SF.

=2,72,24,.65,60,.6,,
   TYPE DOOR DATA FOR TYPE 1 DOORS FOR THIS WALL, IF ANY.
       SAME DATA AND ORDER AS FOR WINDOWS.
=2,78,42,1.09,250,.8,1.50,.02
   TYPE DOOR DATA FOR TYPE 2 DOORS FOR THIS WALL, IF ANY.
       SAME DATA AND ORDER AS FOR WINDOWS.
=0,0,0,0,0,0,0,0
   TYPE DOOR DATA FOR TYPE 3 DOORS FOR THIS WALL, IF ANY.
       SAME DATA AND ORDER AS FOR WINDOWS.
=0,0,0,0,0,0,0,0
   IS WALL  2 THE LAST WALL
=NO
   TYPE WALL DATA FOR WALL  3 --HEIGHT IN FEET,WIDTH IN FEET,U FACTOR,
      BELOW GRADE DESIGNATOR(1. IF BELOW GRADE, 0. IF ABOVE GRADE)
      WALL ORIENTATION DESIGNATOR(1. IF SOUTH FACING,2. IF EAST
      OR WEST FACING, 3. IF NORTH FACING).
```

Figure 1. Portion of terminal keyboard listing from data input activity, HEATCOST program.

```
TOTAL BTUS LOST ARE AS FOLLOWS--
     WALLS                          194573826.
     WINDOWS                         17799654.
     DOORS                           11456032.
     FLOOR                           26207056.
     ROOF                           129266301.
     ROOF ENCLOSURES                  3931058.
     WINDOW CRACKS                   25576448.
     DOOR CRACKS                     16543204.
     ROOF ENCLOSURES CRACKS          11793175.
     SOLAR HEAT GAIN (MINUS)           358624.

          TOTAL                            436788128.

TOTAL ESTIMATED COSTS ARE AS FOLLOWS--
          TEMPORARY ENCLOSURES INSTALLATION      $  349.96
          HEATING                                $ 1048.29
          TEMPORARY ENCLOSURES MAINTENANCE       $   31.50

               TOTAL                               $ 1429.75

   DO YOU WISH TO REVISE SOME INPUT DATA
=YES
        SAMPLE 80X100 BUILDING FOR BENNETT PAPER

   ARE THERE ANY WALL DATA REVISIONS FOR WALL  1
=NO
   ARE THERE ANY REVISIONS TO WINDOW TYPE 1 DATA IN WALL  1
=YES
   TYPE WINDOW DATA FOR TYPE 1 WINDOWS FOR THIS WALL, IF ANY --
      NUMBER, HEIGHT IN INCHES,WIDTH IN INCHES, U FACTOR,
      CRACK INFILTRATION FACTOR(CF/HR/FT OF PERIMETER),
      SOLAR RADIATION TRANSMISSION RATIO(BETWEEN 0. AND 1.0),
      INSTALLATION LABOR COST/SF,MATERIAL COST/SF.
```

Figure 2. Heat loss and cost summary, and beginning of data input revision, HEATCOST program.

Eight of these nine projects were monitored closely, to document the methods used to provide temporary enclosures and temporary heating while under construction. Table 2 lists, in summary fashion, the results of this portion of the study. In general, most of the buildings were fairly well "closed in" prior to the winter period, and the primary construction activities during the winter consisted of interior installation and finish work. Temporary enclosures for most projects were limited to windows and doors, and to larger openings left open temporarily to allow the movement of materials and equipment. In all cases, the projects utilized temporary heating systems for at least a portion of the winter, rather than relying on the buildings' permanent heating systems.

Enclosure methods included lumber frames covered with polyethelene sheeting for covering doors and windows, plywood for doors and other openings (some of which was covered with insulation) and pipe scaffolding covered with polyethelene sheeting. The use of both plain and fiberglass-thread reinforced polyethelene was observed.

In contrast to these rather crude enclosure methods, contractors utilized some rather sophisticated temporary heating techniques. Equipment ranged from the small, easily portable heater of the "Master" variety, rated as low as 300,000 btu per hour to the 2,000,000 btu per hour "Tioga" oil fired furnace. This latter type of furnace, though portable, is generally placed in one location for the duration of the project.

The 1976-77 Fairbanks winter was considerably warmer than normal. For the three month period between December and February, the mean temperature was about 13.5°F (7.5°C) above normal. Nonetheless, some valid conclusions regarding wintertime construction activities were drawn as a result of the study. Contractors do work in the winter. They attempt to finish the exterior shell prior to the onset of winter and then work mainly on interior installation and finish work during the winter. Temporary enclosures are generally limited to doors, windows, and other openings, and temporary heating methods are normally used on all projects.

ROOF CONSTRUCTION DURING THE WINTERTIME

I. Interior City Branch Study

One of the projects surveyed during the 1976-77 winter was approached in a somewhat different manner than most projects that winter. (5) The contractor erected a shell of polyethelene around the entire project and worked inside for a portion of the winter, erecting foundations and exterior shell. After the exterior walls were completed, the enclosure was removed and roof framing, insulation and roofing were installed

Table 2. Temporary Protection Methods, Fairbanks Area, Winter 1976-77

PROJECT	TEMPORARY ENCLOSURES	TEMPORARY HEATING
Bentley Mall	Polyethylene-frame enclosures for windows and doors. Some plywood doors. (Permanent walls and roof were constructed prior to winter period).	Six (6) "Powermatic" oil-fired furnaces. 450,000 BTU/hr. 4 gal/hr. of #2 diesel fuel. One furnace in each of four buildings; two furnaces in central area.
Borough Library	Plywood and Polyethylene-frame doors. Plywood and insulation on windows during window installation. Polyethylene sheeting on lumber frame at main (southwest) entrance. (Permanent walls and roof were constructed prior to winter period).	Two (2) "Armstrong" oil-fired furnaces. 450,000 BTU/hr. 4 gal/hr. of #2 diesel fuel. Several small "Master" heaters.
Fairbanks Furniture, Inc., Store	Polyethylene sheeting on large entrance doors. (Permanent walls, roof and windows were constructed prior to winter period).	One (1) "Master" heater on first floor.
Fairbanks Medical Clinic	Plywood doors. Polyethylene-frame windows. Some polyethylene on lumber frame for wall enclosures. (Portion of permanent walls and roof were constructed prior to winter period).	Two (2) "Jackson and Church" oil-fired furnaces. 400,000 BTU/hr. 3.6 gal/hr. of #2 diesel fuel.
Federal Building	All permanent including most doors and windows, except for main entrance at northeast corner, which was polyethylene frame and plywood. Garage area covered with polyethylene sheeting. Some doors covered with reinforced polyethylene sheeting.	Two (2) "Master" heaters (1,000,000 BTU/hr.) Two (2) "Tioga" oil-fired furnaces (2,000,000 BTU/hr.)
Interior City Branch	Pipe scaffolding covered with black, fiberglas-thread reinforced polyethylene sheeting. Exterior walls were then erected within this enclosure. Steel (above) was erected after enclosure was removed.	Three (3) "Master" Heaters (385,000 BTU/hr.) (65 gal diesel fuel/hr. total) One (1) "Herman Nelson" furnace
Plywood Supply	Windows of polyethylene-lumber frame. Plywood doors. Openings at both ends covered full-height with polyethylene sheeting and plywood panels. (Balance of walls & roof were completed prior to winter period.)	One (1) "Powermatic" oil-fired furnace. 450,000 BTU/Hr. 4 gal/hour of #2 diesel fuel.
University Mall	Front window portion enclosed with insulated plywood. (Permanent walls and roof were constructed prior to winter period.)	Two (2) Jackson and Church "Flexaire" oil-fired furnaces. 500,000 BTU/Hr. 4.5 gal/hour of #2 diesel fuel. Several small "Master" heaters.

under winter conditions without benefit of temporary enclosures or heating. Since built-up roofing is normally installed at temperatures above freezing, this project was of particular interest and became the subject of a follow-up study.(6)

The roof is that on an addition to the Interior City Branch of the First National Bank of Anchorage, located in downtown Fairbanks. It is a Class 1 insulated, fire-retardant, built-up roof with a one hour building classification, placed over a 1 1/2 in. (3.8 cm) 20-gauge steel deck by H.H. Robertson. The vapor barrier is installed above the deck, over which are placed two layers of fiberglass insulation, one with 1 5/8 in. (4.1 cm) thickness and one with 1 3/16 in. (3.0 cm) thickness. Over the insulation are placed two layers of bituminous woven glass fabric, covered with two layers of 15# (6.8 kg) asphalt saturated asbestos felt. Plain asphalt is used between roofing plies and as surfacing.

This roof was built between January 14 and February 11, 1977. Table 3, entitled "Roofing Construction Schedule," gives details on weather conditions and construction activities during that period. The lowest temperature on a work day during this period was -14°F (-26°C), and the highest temperature was +45°F (+8°C). Although these temperatures averaged considerably higher than normal, conditions during some of the period could nonetheless be considered as "wintertime." It was necessary to suspend operations during four days because of snow. Following snowfalls, the contractor moved snow to completed portions of the roof, eliminated any remaining moisture from the uncompleted section, and continued roofing operations.

Although constructed in the wintertime, the roof has performed satisfactorily to date. The owner reports that no problems were experienced during the 1977-78 winter, including the spring 1978 breakup. In May 1978, the building, including its roof, passed final inspection by the City of Fairbanks Building Department.

II. General Observations

The writer had the opportunity to interview Mr. Edwin Burbeck, of Burbeck Roofing Co., the contractor responsible for construction of the roof at the Interior City Branch Bank. (6) This company has built nine roofs in the Fairbanks area under wintertime conditions, beginning in 1973. Although specifications generally require temperatures to be above +40°F (+4°C), roofing projects have been successfully completed at temperatures as low as -30°F (-34°C).

Mr. Burbeck offered several observations relative to the construction of roofs in wintertime, as follows:

- Labor costs in the wintertime increase by between one-third and one-half, because the workmen move about half as fast, due to bulky clothing; the men warm up about once every hour; and efforts are required to warm roofing materials.

Table 3. Roofing Construction Schedule, Addition to Interior City Branch Bank

		Temperature, °F				
		Average	Maximum	Minimum	Precipitation	Construction Activity
January	14	- 3	2	- 7		Place Vapor Barrier & Insulation
	17	13	25	1		Place Insulation
	18	10	19	0		Place Insulation
	19	- 3	4	-10	Snow-Trace	Place Insulation
	20	- 2	10	-14	Snow-Trace	Begin Roofing
	21	4	11	- 4		Roofing
	24	28	46	9		Roofing
	25	22	30	14		Roofing
	26	26	32	19	Snow-3.1"	No work-snow
	27	26	29	23	Snow-0.1"	Move snow; Roofing
	28	19	23	14	Snow-Trace	Roofing
	31	- 2	0	- 3	Snow-1.4"	No work-snow
February	1	- 3	0	- 6	Snow-9.5"	No work-snow
	2	- 2	2	- 1	Snow-7.3"	No work-snow
	3	2	6	- 2		Move snow to completed half; Roofing
	4	11	24	- 3		Roofing
	7	7	15	- 2		Roofing
	8	8	18	- 2		Roofing
	9	1	8	- 7		Roofing
	10	4	13	- 5		Roofing
	11	- 1	9	-10	Snow-Trace	Complete job; Move out

Temperature and precipitation data from Local Climatological Data, National Weather Service Office, Fairbanks International Airport.

- Whereas normal material wastage is approximately 5%, such wastage is between 10% and 20% in cold weather.

- Additional asphalt is required in cold weather. If the specifications call for 30# of asphalt per 100 sq. ft. (1.47 kg/sq. m), one should use 35 to 40# per 100 sq. ft. (1.70 to 1.95 kg/sq. m) during cold conditions.

- The most important problem encountered in roofing under cold conditions is moisture from snow, mist and fog. In order not to get moisture between plies, it is important to put all

plies down in succession, rather than waiting until the next day to complete the operation.

- It is important to gather the materials and supplies into one place at the completion of a day's operations, and to protect the material in that one place from possible snowfall. Some materials should be kept inside at night and taken out in the morning for use, thus preventing the need to warm the cold material prior to application.

- A successful sequence is the following: (1) Insulate the entire roof; (2) Cover the entire roof to protect from possible snow; (3) Complete a small area of roofing, prior to moving on to another area; (4) If snowfall occurs, push the snow from the area to be worked onto a completed area.

- Roofing material required for parapet walls cannot be applied successfully in the wintertime. Instead, a temporary material should be used to make a watertight covering, with the heavier permanent material applied after the end of the winter.

- Metal flashing should never be applied in the wintertime.

- Always install interior penetrations in their final, permanent condition, in conjunction with cold weather roofing projects.

- Roof drains can be set in the winter, using an excess of asphalt, and the drain is then tightened into final position after the end of the winter.

- Roofing application inside an enclosure is generally not successful, because of the accumulation of toxic fumes and because mops and other tools tend to strike the enclosure during use.

- Cooperation and coordination with the general contractor is essential for a successful cold weather roofing operation.

SUMMARY AND CONCLUSIONS

Contractors and owners, spurred by recent accelerating trends toward development of the North, are increasingly realizing the possible benefits of wintertime construction. For the contractor, benefits include utilization of idle equipment and manpower and the receipt of income sooner than if he delayed construction work into the summer period. For the owner, his project will be available more quickly, and the total construction cost may be less than if the project were shut down during the winter and resumed with the coming of warmer weather. For the construction worker, construction work in the winter is much appreciated, especially when compared to a winter of forced idleness.

This paper has sampled some recent trends in the construction of buildings in the Fairbanks, Alaska, area during the wintertime. It reported on a questionnaire survey of Alaskan contractors, which indicated that some contractors believe that some types of construction activities can proceed at temperatures as low as -70°F (-57°C). A case study of a typical wintertime building project in Fairbanks, which was protected with temporary enclosures and heated by two temporary heating systems, was reported; from this study, it was concluded that a cost estimate of approximately two percent of total construction contract price should be allocated to the provision of a temporary enclosure and heating scheme for wintertime protection.

The paper then described a computer program that will allow the construction contractor or owner to estimate the cost of providing temporary enclosures and heating for buildings under construction in cold regions and to compare the cost estimates of various temporary protection schemes. A survey of nine building construction projects that went forward during the 1976/77 winter in Fairbanks was also reported. This survey indicated that contractors generally use rather crude methods for providing temporary enclosures for their projects, but that heating methods, using relatively large, stationary units, are becoming more sophisticated.

Finally, the paper reported on roof construction in the wintertime, citing a case in which a build-up roof was constructed at temperatures as low as -14°F (-26°C). Also, some general observations and suggestions by a veteran Alaska roofing contractor on roofing construction in the wintertime were presented.

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PREFABRICATED LOW-STORY CONSTRUCTION WITH THE USE OF LIGHT MATERIAL STRUCTURES FOR THE FAR NORTH

by V.V. Sudakov[1]

INTRODUCTION

The intensive conquest of the territories of the Far North, distinguished as a rule by complex macroclimatic and permafrost conditions, has necessitated moving in a new direction of construction: To the wide use of light buildings.

Experience in planning and construction, and also economic research, have shown that buildings of light construction using steel, aluminum, foam plastic and other efficient materials correspond to particularly inaccessible regions to the greatest degree. These buildings satisfy the basic requirements (architectural expressiveness, provision for comfort, agreement with construction norms and regulations, and so forth) for remote northern regions, namely:

1. maximum reduction in the weight of prefabricated elements, and the building as a whole, makes it possible to transport them by different means and to assemble them with little equipment

2. durability and reliability during severe climatic conditions

3. maximum degree of prefabrication without requiring "wet" processes at the point of assembly along with a significant reduction in the difficulty and time of construction

4. multiple reusability with rapid retransportation to new points of utilization.

It should be noted that prefabricated wooden houses of different types of construction using efficient heaters satisfy these requirements. A number of fairly highly developed and technically well equipped lumber mills have mastered their production. However, in many cases they prove to be unsatisfactory for the northern climatic zone with respect to transportation, prefabrication and thermophysical parameters. The delivery of wooden buildings to construction sites is by means of complicated transportation systems, and the necessity for frequent relocation causes significant damage to the buildings, thus reducing their reliability.

TYPES OF PREFABRICATED BUILDING SYSTEMS

Light structures of efficient materials may be used both in stationary, fully prefabricated buildings, as well as in relocatable, mobile buildings

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usually called inventory buildings. These include: travelling (very mobile) buildings, fully prefabricated in the factory, equipped with an undercarriage; container buildings of unit construction, or put together from elements (each of which is an independent part of the building) complete with built-in engineering and production equipment; and prefabricated collapsible buildings made up of separate linear flat independent elements.

It is expendient to use mobile homes ("trailers") when building linear installations (roads, power lines, pipe lines, and so forth) and for small settlements (25-150 people) remaining in one place from several days to a month. They also find use in a modified form for individual "nomadic" professions (reindeer herders and so forth). Thus, a number of buildings for reindeer herders (for 2-4 people, using plastics and efficient materials), which are moved by reindeer teams or mechanically, have been created in LenZNIIEP.

In most cases mobile homes are not promising for the creation of residential and public complexes in inaccessible regions for the following reasons:

1. It is practically impossible to create residences corresponding to contemporary requirements with independent trailers

2. In comparison with block container buildings, mobile homes require 1.5-2.0 times more metal per square meter of area because of the reinforced undercarriage, this metal in effect going to waste when the trailer is standing in one place

3. In roadless conditions, which are characteristic of northern regions, it is impossible to move mobile homes "on their own power" for significant distances; as a rule, special means of transportation are required for this.

It is expedient to use container buildings in areas of the Far North for mobile settlements, including duty and pioneer settlements, located in one place from several months to several years. The recommended population of the settlements is up to 300 persons.

SMALL INDUSTRIAL INSTITUTIONS AND EXPANSION COMMUNITIES

Prefabricated collapsible buildings can be used for settlements of up to 3,000 persons, including base villages with a comparatively long operation in one place (more then 3-5 years).

A list of low-story prefabricated buildings for settlement of different sizes and different periods of operation (9 types in all) is being developed at the present time.

In developing the list, municipal planning conditions were considered: the place of a town in the general system of settlements, its functional purpose, and the degree of connection with other towns and cities. On the other hand, the development of the list involves providing for the necessary set of buildings in order for each type of settlement to function. Sets of

residential and public buildings for different purposes and for each characteristic type of settlement were compiled in order to determine the necessary list of buildings according to types and size (capacity).

Living area standards were taken in accordance with VSN 12-73: in residential buildings of the apartment type (9 m^2), in hostels (6 m^2), and in mobile homes (4.5 m^2) per person (for brief stays or pioneer villages).

The standards for providing settlements of different types with cultural services are taken in accordance with the "temporary instruction for planning inventory public buildings," VSN 34-77.

The conditions of the different climatic regions are considered for each of the buildings.

Because of the fact that the overall dimensions of mobile homes are limited by the transportation facilities, it becomes necessary to minimize their accommodations. This creates significant limitations in determining the capacity and choice of designs.

The requirements imposed on stationary buildings are extended to prefabricated collapsible buildings. The difference is in the structural solution of joints, in the assembly time, the provision for disassembly for moving, the light construction, and so forth.

The capacity of public buildings is determined by the size of the different settlements. The following groups are provided for in the list of public buildings:

Educational buildings

a) preschool buildings
b) general education schools

Medical buildings
Commercial buildings
Theatrical buildings
Sports buildings
Administrative buildings.

The creation of a large number of fairly small industrial institutions - oil and gas fields, compressor stations, supply bases, transfer stations and ports with a limited period of functioning - is characteristic of the initial stage of development of new northern regions. Many drilling, surveying, and construction crews often change the place where they work. Therefore it becomes necessary to build small temporary towns with populations of from ten to several hundreds of persons.

In these conditions, the creation of comfortable living conditions and a high level of service for a mobile contingent of workers is an urgent problem.

Two aspects, social and technological, arise in the solution of this problem.

On the social plane, Soviet architects consider both the nature of the work as well as the specific features of the sex, age, and family composition of the population when planning residential and public buildings and the settlements themselves for a mobile contingent.

Therefore, the existing standard documents and plans relating to the service systems are being corrected. In particular, the arrangement and size of kindergartens and nursery schools is being corrected. And an increase in the role of boarding schools in a number of large settlements and the elimination of schools in certain other types of settlements are being planned. Because of the increased percentage of young people and single people in these settlements, the number of sports facilities and places for passing leisure time are being increased.

On the whole, it is characteristic that the social services in towns are set up according to the principle of greater generalization and increased significance of collective forms of leisure, recreation, and social services as a whole.

Great attention is given to the rounded and harmonic development of personality, for which, in addition to cultural and recreation institutions in these villages, there are plans for increasing the number of facilities for hobbies, and raising the level of correspondence types of middle and higher special education.

All of these problems are reflected in scientific studies carried out in the Soviet Union.

Normative documents, the temporary recommendations and instructions for planning residential and public inventory buildings, have been developed. A methodological handbook for planning inventory buildings has been written and the experience in planning these buildings in the leading institutions of the country has been summarized.

In spite of the fact that towns made up of low-story prefabricated buildings, for the most part, are of a temporary nature, it is intended that they be fully equipped. Therefore, a number of institutes are carrying out extensive research and planning studies.

The technological aspect of the problem is connected with a number of specific features distinguishing buildings for small and temporary settlements from stationary buildings. These differences are connected first of all with the rigid requirements for transportation of the buildings and assembly with the use of limited equipment.

LIGHTWEIGHT MATERIAL USE

One feature of light buildings made of efficient materials is that they are included in the fifth degree of fire resistance. This leads to a number of requirements, in particular:

1. The height of the building is limited to one or two stories

2. The maximum area is 800-1200 m^2, which limits the capacity of the building.

3. The length of service lines and roads is increased when settlements are constructed with these buildings. All of this requires efficient planning of settlements.

Another characteristic feature of low-story prefabricated buildings made out of efficient materials is the fact that they must be assembled in short periods of time with minimum use of labor. Buildings assembled from repaired functional elements, provided with all necessary equipment, correspond to this condition to the greatest degree. This practice is becoming more and more common in many countries: in the Soviet Union, the United States of America, Canada, Sweden, and Norway. This, in turn, requires a new approach to the architectural design of buildings, when it is necessary to design buildings of individual functional units: a living unit, a kitchen unit, and a lavatory unit, combining living facilities with kitchen and sanitary equipment. In order to increase the level of prefabrication of such units and buildings as a whole, it is necessary that they be standardized, reducing the number of dimensions and obtaining maximum universality.

The construction system of fully prefabricated and prefabricated collapsible buildings was worked out in stages, beginning with construction buildings and ending with the development of large specialized buildings. The perfection of construction solutions proceeded from stage to stage.

Buildings assembled from elements made directly at the construction site, or at local nonspecialized plants, were designed in the first stage. These buildings usually were designed according to the panel-frame system, where 3-layer aluminum panels were hung on a standard frame of angle iron. The panels are insulated with PSBS polyurethane foam, FRP-I. and mineral wool. In recent years, several dozen such buildings have been constructed in the Arctic and Antarctic, including: service-residential complexes, permafrost laboratory buildings, 2-story hostel buildings for 50 people with a dining room for gas collection points, and so forth.

Experience in constructing these buildings has shown that they have advantages over those made of traditional materials in cost, labor, and materials. In addition, with respect to these criteria, construction buildings are inferior to buildings made of elements manufactured on high performance equipment; their use is justified only by an insufficient number of specialized factories for manufacturing light structures. Nonetheless, the construction of these buildings made it possible to check them out thoroughly in natural conditions and to establish scientific grounds for moving to their commercial production.

The water, heat, and electricity systems of the buildings under consideration differ little from traditional buildings.

The necessity for mechanical ventilation in northern regions complicates the construction of the buildings, and requires special measures for reducing the acoustic influence of the ventilation units on living areas.

In view of the small number of stories, the construction of a service floor is economically unjustified, and a crawl space, the minimum height of which is determined by the possibility of laying supply lines, is created instead of it.

Container buildings differ from completely prefabricated and prefabricated-collapsible buildings first of all by the fact that they consist of container units manufactured completely in factories.

RESIDENTIAL CONTAINER UNITS

The optimum area of container units used as residential compartments for 2 to 3 people is determined by the residential area standards, figuring on 4.5 to 6 m^2 per person, and equal to 15 to 18 m^2. The height of the compartments (finished), is 2.5 m. It may be reduced to 2.4 m for appropriate reasons and with the agreement of sanitation officials.

The outside dimensions of the cross section are limited by the railroad clearance. Because of the fact that it is necessary to match the face and sides of units in putting together buildings it is efficient to use a ratio of the width of a unit to its length equal to 1:2.

The use of limited equipment in assembling container buildings limits the weight and dimensions of them. The following dimensions best correspond to the above mentioned requirements: width - 3.0 m, length - 6.0 m, height - 2.85 m.

The structural designs of container units may be divided into three groups: frame-cover, panel, frame-panel. The latter is widely used both for containers and for fully prefabricated and prefabricated collapsible buildings.

Steel is the most acceptable material for use in the support frame of fully prefabricated, prefabricated collapsible, and also container and mobile buildings. It provides the necessary strength and rigidity of construction with a comparatively low weight. The use of a frame of aluminum sections, in spite of the better weight indices, significantly increases the cost of construction. The use of wooden frames for low story buildings is more economical. However, in conditions where buildings are repeatedly moved, the connections of the wooden elements are significantly inferior to steel elements in reliability and rigidity. In addition, a wooden frame requires additional protection from the point of view of fire safety.

For a 20-30% saving of steel, efficient thin-walled bent steel sections are being used more and more in building frames.

The surroundings require the most material and labor. For light buildings, most frequently, they are made in the form of layered panels of small or large size. Soviet experts give preference to the latter large panels with "room-size" and "unit-size" dimensions. They make it possible to reduce the number of joints - the weakest, most vulnerable points in the walls.

Different plastic foams and also mineral wool sheets with synthetic binding are used in the USSR for insulating the middle layer of the panels. Mineral wool sheets are used in the wall panels of buildings having high fire safety requirements (children's and medical institutions). Of the plastic foams, in recent years preference has been given to phenol plastic foams, which, in addition to the simplicity of production, are capable of supporting significant loads, are very long lasting, and fire resistant.

Different sheet and plate materials (steel, aluminum, asbestos cement, wood particle and wood fiber boards, veneer,and so forth) are used for covering layered panels. All of them have their advantages and disadvantages. A comparative technological analysis of panels with different coverings for a large series of prefabricated collapsible and container buildings (the delivery of which will be performed by a large specialized combine) demonstrated the advantages of 2-walled aluminum coverings.

The insufficient assurance of long term reliability of the connection between plastic foams and coverings, and also the necessity of using mineral wool board insulators in certain cases, usually makes it necessary to use framed panels. Wooden rods and water resistant and bakelite treated plywood, which have low heat conductivity (which is very important in the case of low negative temperatures) and fairly high mechanical strength, are most acceptable for panel frames. Studies on the organization of the mass production of plywood channel sections are being carried out at the present time. A special mechanism for connecting these sections with aluminum coverings by means of rolling the latter have been developed. This makes it possible to do away with rivets and wood screws, and also with additional connecting angles, which significantly reduces manufacturing difficulties.

DESIGN CRITERIA OF STRUCTURAL ELEMENTS IN FROZEN CLIMATES

The development of sections made of different plastics is being carried out at the same time.

The foundations for light low-story buildings are made by different methods: by means of grading the surface, laying beams, using stock metal and bracing and structural plates and, finally, piling made of pipes of small diameter sunk deep into the ground. The particular solution used depends on the permafrost conditions, the period of operation in one place, and also on the functional purpose of the building. A basic trend in the future development of the production of light buildings of steel, aluminum, plastic, and other efficient materials is the creation of specialized house construction combines for manufacturing these materials in order to perfect and standardize house construction. The construction plant for manufacturing a series of buildings using aluminum and other efficient materials in Sayanogorsk was the first such combine. The creation of shops for preparing 3-layer panels on domestic high production equipment and also the assembly of all elements at the plant itself (or at rear bases in territorial administrations), with the organization of assembly crews, also is envisaged.

The list of building types, consisting of fifty items, includes a complete set of residential, public, communal, and factory buildings. The buildings of such an extensive list are distinguished by the height of the story and the width of the support structures, and the dimensions of the openings. Therefore, it was necessary to carry out extensive work in standardizing structural elements, providing for the necessary flexibility in dimensions.

A technical economic analysis showed that in comparison with buildings made of traditional materials the cost of light buildings is 30% lower and reduces construction labor by more than two times. The operating expenses (repair, heating, and so forth) also are reduced.

CONCLUSIONS

The reduction in the weight of the buildings implies a comparatively low consumption of steel. For panel-frame residential and public buildings it amounts to 50-53 kg, and block container buildings 52-55 kg per m^2 of total area, including the 36-33 kg per m^2 of area for the frame of the buildings.

The comfort of the buildings developed is an additional factor in the efficiency since an improvement in living conditions leads to a reduction in the turnover of the labor force, securing qualified personnel, and in the final analysis, an increase in labor productivity. At the present time it is difficult to evaluate it in a cost equation. Such an evaluation may be performed in the process of the utilization of these buildings by means of conducting special economic research.

SOLAR AIR HEATING SYSTEMS FOR HIGH LATITUDE, COLD CLIMATES UTILIZING EVACUATED TABULAR SOLAR COLLECTORS

By Dan S. Ward[1]

ABSTRACT

Solar heating systems designed specifically for high latitude (55° to 70°) and cold climates may offer a means of conserving those non-renewable energy resources used in providing heat to building construction sites. Complete solar systems can be prefabricated in transportable modules for quick installation on site and can provide low grade heat (30-60°C) (86-140°F) for a variety of construction-related heating requirements.

This paper describes a solar heating system capable of providing heat to a building during construction and which is designed for use in high latitude and cold climates. The system utilizes advanced solar air heating evacuated tubular collectors to provide heat directly to heating loads or to a pebble-bed thermal storage subsystem for later use.

Critical to the design feasibility of a solar system for these cold climates is the utilization of evacuated tubular solar collectors, capable of providing heat at low solar insolation rates (I) and at high differentials between the collector operating and the ambient temperatures (ΔT), i.e., for conditions of $\Delta T/I = 0.5\ °C \cdot m^2/watt$. Calculations are presented in order to demonstrate the technical feasibility of such solar systems.

INTRODUCTION

Construction of buildings in high latitudes (55° to 70°) and cold climates often requires low grade heat at temperatures of 30 to 60°C in order to simplify construction procedures. Such heating may be utilized for improved working conditions of personnel, for increased troublefree operation of equipment, and/or for a variety of construction, fabrication, and installation practices.

The availability of conventional heating sources, however, may in some areas be strongly limited because of high fuel costs and/or difficulties in transporting heating fuels to the construction site. In such cases where conventional heating is not readily available, or is severely restricted in its use, alternative heating methods may be employed.

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It is worthwhile to consider whether or not solar heating systems could be utilized as an alternative to conventional heating systems using nonrenewable fossil fuels. The use of solar energy as a heat source in high latitude, cold climates may appear questionable at first glance because of the inherent low solar insolation and low ambient temperatures encountered. However, low cost, high performance evacuated tubular solar collectors are now available which are capable of useful heat collection at low solar insolation rates (I) and high temperature differentials between the collector and ambient (ΔT), e.g., $\Delta T/I = 0.5$ $°C \cdot m^2/watt$. While the capability of solar to provide for particular heating requirements at specific construction sites will necessitate individual consideration, solar systems utilizing these advanced solar collectors may, in many applications, offer a realistic alternative to conventional heating.

EVACUATED TUBULAR SOLAR COLLECTORS

The concept of an Evacuated Tubular Solar Collector (ETSC) was first proposed by Speyer [1]. Since then numerous designs have been proposed, including those under development by Corning, Owens-Illinois, General Electric, and Phillips. Many of these designs utilize water or water/ethylene glycol mixtures as the solar collector heat transfer liquid. For rather obvious reasons, these designs cannot be recommended for high latitude and cold climate applications.

However, both Owens-Illinois and General Electric have ETSC designs which utilize air as the heat transfer medium. Figure 1 shows a schematic view of the Owens-Illinois ETSC [2]. The heat transfer fluid (air) travels in a U-shaped pattern within the two interior tubes. The inner-most tube is a metal tube while the outer two tubes are composed of high strength glass. Between the outer and middle tubes, a vacuum of 10^{-3} to 10^{-4} torr virtually eliminates conduction and convection heat losses. The outer surface of the middle tube acts as the solar absorbing surface. This absorber surface is a selective surface with a high absorptivity for solar radiation ($\alpha \approx 0.85$) and a low emissivity for thermal radiation ($\varepsilon \approx .07$).

Figure 2 shows a schematic view of two ETSCs manifolded together and integrated with a common duct for a complete array of solar collectors [3]. (Figure 3 shows a complete solar collector array of ETSCs installed on CSU Solar House III.) Air enters the inlet duct, flows into an evacuated tube between the two inner tubes, returns along the innermost tube into a second evacuated tube, exits the second tube by passing between the two inner tubes, and returns to the outlet duct. Each pair of evacuated tubes is in series with individual pairs of evacuated tubes of the array being in parallel flow.

The performance of solar air heating evacuated tube collectors has been demonstrated to be excellent. Experimental measurements with these

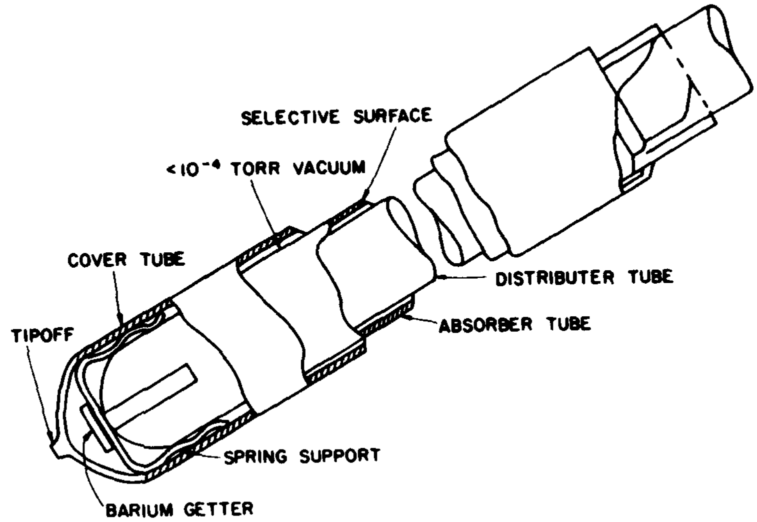

Figure 1. Schematic view of the Owens-Illinois evacuated tubular solar collector.

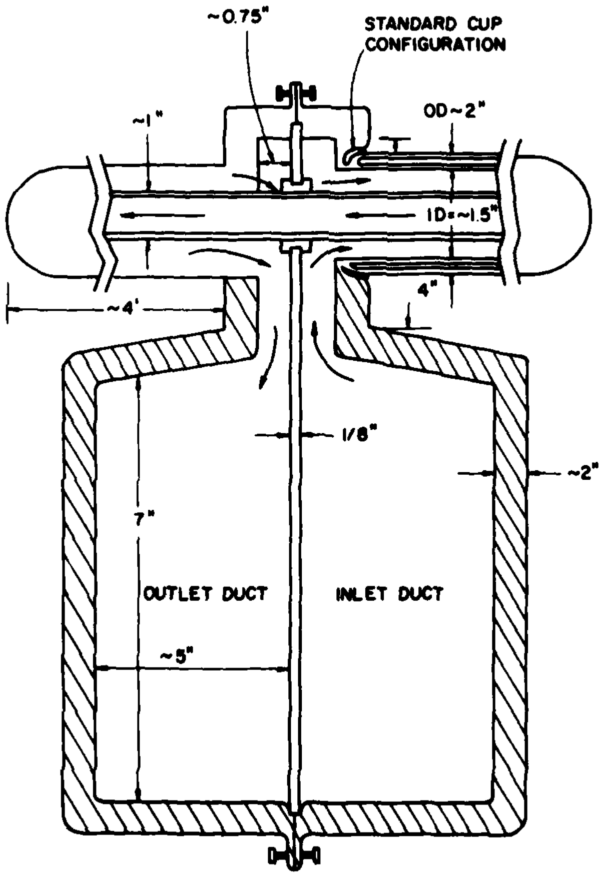

Figure 2. Two ETSCs manifolded together and integrated with a common duct.

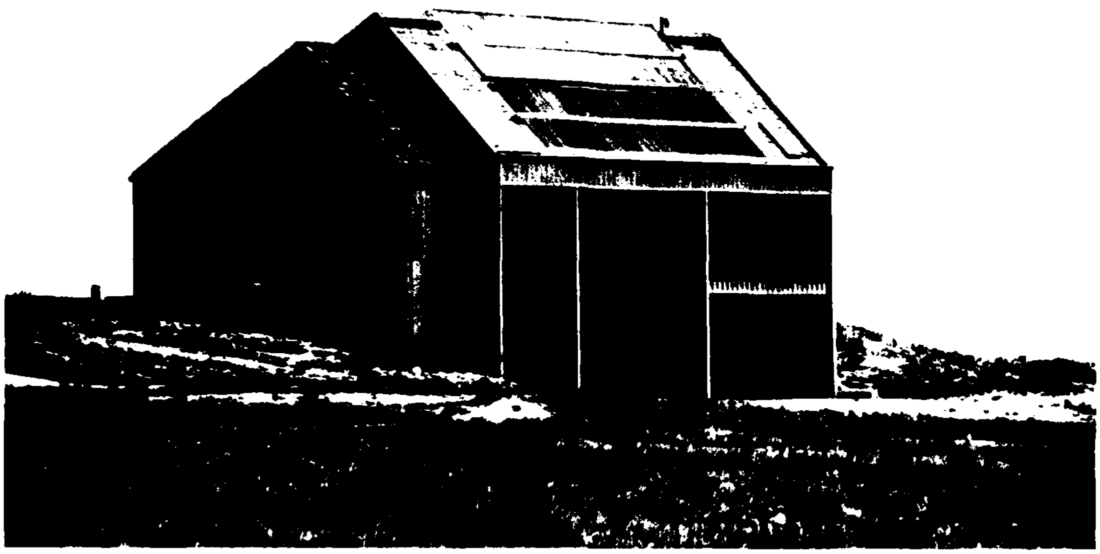

Figure 3. Complete array of solar collectors (Solar House III).

units by Moan [2] indicate collector efficiencies of 40 percent for collector temperatures of 70°C, ambient temperature of -20°C, and mean daily solar insolation rates of less than 15 $MJ/m^2 \cdot day$. For better solar conditions, i.e., mean daily solar insolation greater than 15 $MJ/m^2 \cdot day$, the collector efficiency averaged 50 percent. For lower temperature differentials ($\Delta T \approx 50°C$) and lower daily solar insolation conditions ($\bar{I} \approx 4$ $MJ/m^2 \cdot day$), Jacobsen [3] has reported daily solar collector efficiencies of 30 percent.

SOLAR AIR HEATING SYSTEMS

Integration of the ETSCs with solar air heating systems is a straightforward procedure. These advanced solar air heating collectors can be used to supply heat directly or can be incorporated into a complete solar heating system utilizing thermal storage. Figure 4 is a schematic view of a solar air system designed to provide space and domestic hot water (DHW) heating for a building. Such a design could be easily modified to suit particular construction site applications.

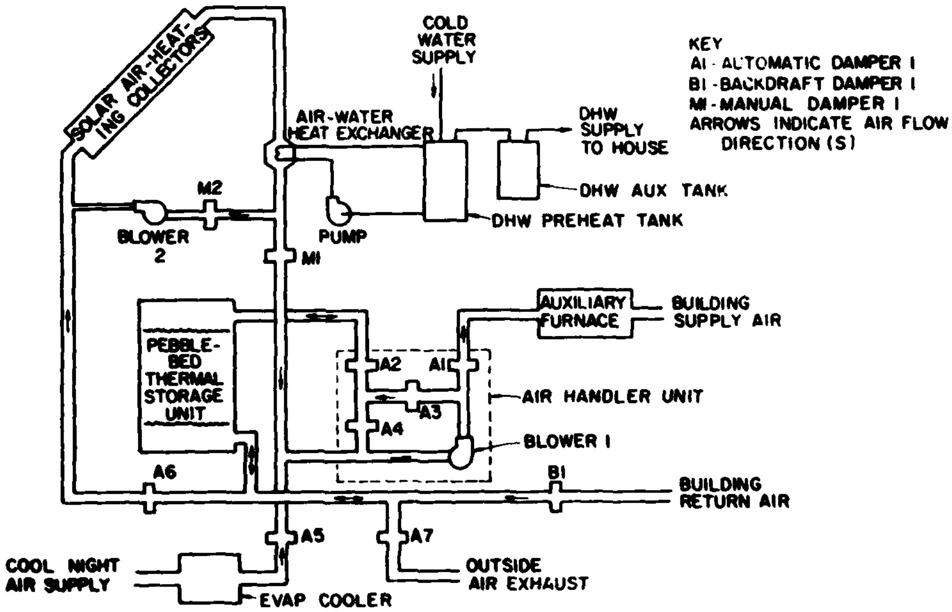

Figure 4. Schematic view of solar air system.

An important component of an air heating system using solar energy is the thermal storage unit. The use of a pebble-bed storage unit is particularly suited for cold climate applications in conjunction with solar air heating systems. Figure 5 shows a sketch view of a typical pebble-bed storage unit. The pebble-bed uses hard, rounded rock with nominal diameters of 4 to 6 cm, with air plenums at the inlet and outlet. An important advantage of the pebble-bed thermal storage is the inherent low heat loss coefficient of the surface area of the unit. This is due in part to the very poor heat conduction through the pebble-bed when the system is not collecting or delivering heat. On the other hand, the very large surface area of all the rock allows for excellent heat transfer whenever air is forced through the bed.

These heat transfer characteristics also allow for important temperature stratification within the thermal storage unit. Solar heated air at 60°C will transfer most of its heat to the pebble-bed as it passes through the first half-meter. Thereafter the air is essentially at the original temperature of the pebble-bed and is returned to the collector. When heating from storage, air is forced through in the opposite direction, so that the air is heated to its highest temperature (in this case 60°C) just prior to its leaving the pebble-bed.

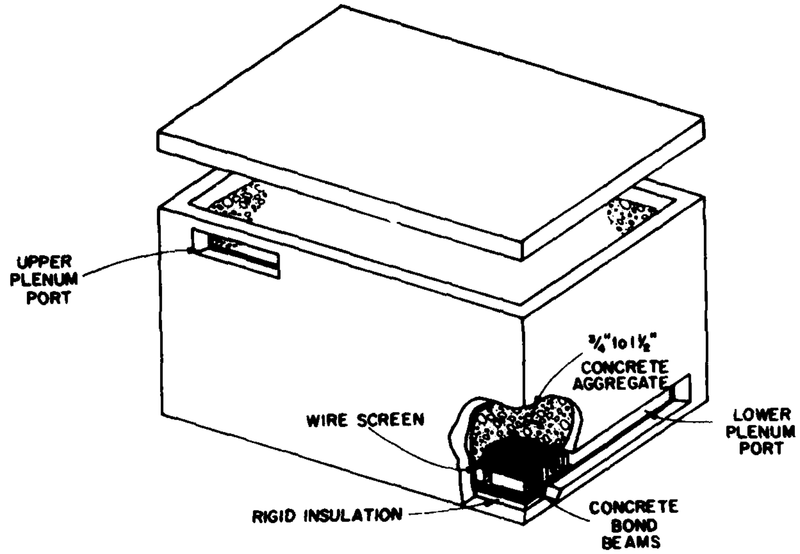

Figure 5. Pebble-bed storage unit.

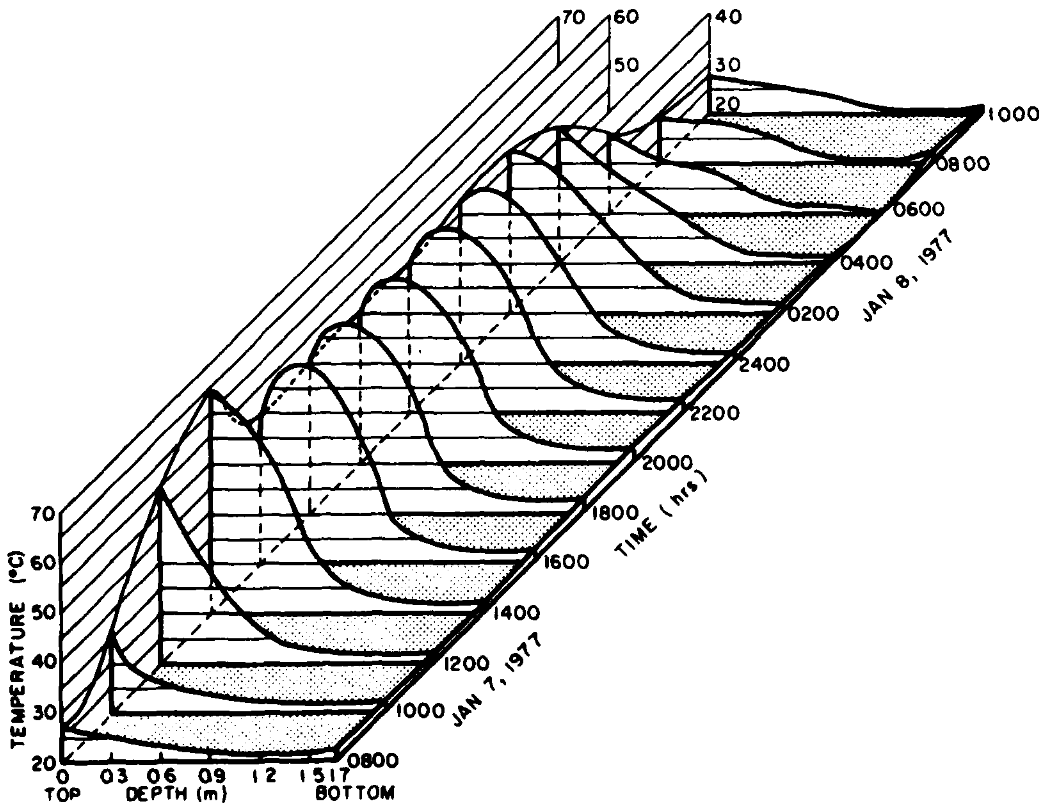

Figure 6. Temperature profiles in the pebble-bed during peak heating season.

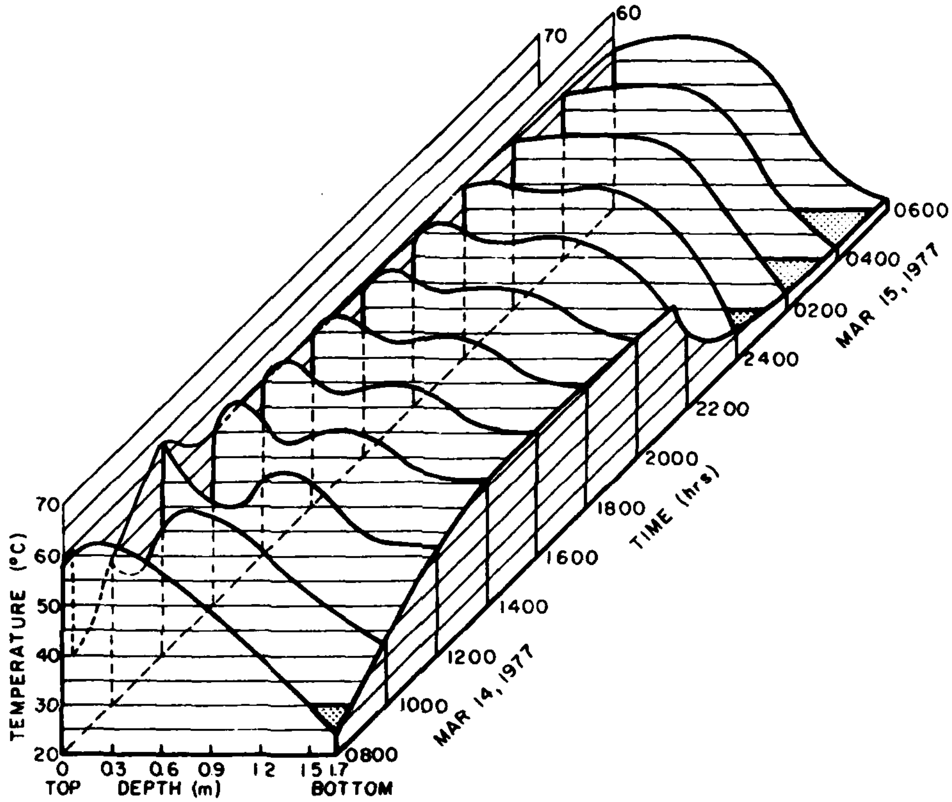

Figure 7. Temperature profiles in the pebble-bed during the end of the heating season.

Figures 6 and 7 show typical inlet temperature profiles in the pebble-bed during different hours of the day. In each case solar heated air enters the top of the pebble-bed. The pebble-bed storage is an important part of the solar air heating system and provides for higher collector efficiencies because of the relatively low collector inlet temperature afforded by the temperature stratification.

TECHNICAL FEASIBILITY

The technical feasibility of a solar heating system as described above must be based on realistic expectations of solar conditions and performance. Table 1 details the climatic conditions of Fairbanks, Alaska, and can serve as an example of a high latitude cold climate. In Table 1, $\overline{I}_H$ is the monthly average daily total radiation on a horizontal surface, $\overline{K}_t$ is the ratio of $\overline{I}_H$ to $\overline{I}_o$, where $\overline{I}_o$ is the monthly average daily extraterrestrial radiation on a horizontal surface, and t_a is the monthly average daytime ambient temperature.

Table 1. Monthly climatic conditions for Fairbanks, Alaska (latitude = 64° 49' N).

Month	$\bar{I}_H$ (MJ/day·m^2)	$\bar{K}_t$ (dimensionless)	$\bar{t}_a$ (°C)
January	0.75	0.639	- 21.7
February	3.22	0.556	- 17.6
March	9.78	0.674	- 10.6
April	16.83	0.647	0.1
May	20.53	0.546	10.3
June	22.40	0.529	16.9
July	19.35	0.485	17.7
August	14.18	0.463	14.6
September	7.95	0.419	8.4
October	3.68	0.416	- 1.3
November	1.18	0.470	- 14.7
December	0.23	0.458	- 21.4

Taken from Liu, et al [4]

From these data we see that an air heating ETSC might be utilized at reasonable efficiency in providing heat nine months out of the year (November, December, and January being excluded). To investigate this quantitatively, it is necessary to convert the solar radiation data in Table 1 to values for solar insolation on a tilted surface. We may then define $\bar{I}_T$, the monthly average daily total radiation on a tilted surface, with the equation:

$$\bar{R} = \frac{\bar{I}_T}{\bar{I}_H}, \tag{1}$$

where $\bar{R}$ is just the ratio of the daily radiation on a tilted surface to that on a horizontal surface. Klein, et al. [5] have provided means for calculating $\bar{R}$, i.e.:

$$\bar{R} = (1 - \frac{\bar{D}}{\bar{I}_H})\, \bar{R}_b + \frac{\bar{D}}{\bar{I}_H} (1 + \frac{\cos s}{2}) + \rho\, (\frac{1 - \cos s}{2}), \tag{2}$$

where:

s is the tilt of the collector from the horizontal (degrees),

ρ is the ground reflectance (typically 0.2 for dry areas and 0.7 for snow).

and $\overline{D}/\overline{I}_H$ is determined from the expression:

$$\frac{\overline{D}}{\overline{I}_H} = 1.3903 - 4.0273\,\overline{K}_t + 5.532\overline{K}_t^{\,2} - 3.100\,\overline{K}_t^{\,3} + \ldots . \qquad (3)$$

Here, $\overline{R}_b$ is the ratio of the daily direct (beam) radiation on a tilted surface to that on a horizontal surface, and is a trigonometric function of the latitude, collector tilt, declination (time of year), and sunset hour angle.

Using the earth's declination (δ) of the mid-point of each month, we can calculate $\overline{R}_b$, $\overline{R}$, and $\overline{I}_T$ for each month. Table 2 shows the results of these calculations for two collector tilts for Fairbanks, Alaska. The vertical collector optimizes the system for the winter season and provides for a simpler installation. The collector tilt, equal to the latitude, optimizes the collector system for the March-April and September-October periods. The latter angle virtually writes off its use in December and, to a lesser extent, November and January, and therefore, attempts to provide heating for a nine-month construction period.

Evaluation of the data in Table 2 in conjunction with the performance of the ETSC, already discussed, indicates that collector efficiencies of 50 percent would be possible for six to eight months and as high as 40 percent for five to three months. Only December appears to be unrealistic in terms of energy collection.

Unfortunately, these data are based on daily averages and the wide variations in solar availability during the year at high latitudes requires a more careful approach. It is, in fact, necessary for us to consider the average hourly variations in the solar availability during different times of the year. In addition, it is also important to consider the minimum solar insolation rate necessary for the useful collection of solar energy.

Using procedures devised by Liu and Jordan [6], we can obtain values for the hourly radiation on a horizontal surface as a function of the daily radiation, the mid-point of the hour before or after noon, and the sunset hour angle (alternatively the length of the day). The results of this analysis for the case where the tilt of the collector equals the latitude are shown in Table 3.

Table 2. Calculation of $\overline{I}_T$ monthly values for Fairbanks (latitude = 64° 49' N).

Month	δ	$\frac{\overline{D}}{\overline{I}_H}$	Tilt = Lvertical			Tilt = Latitude		
			$\overline{R}_b$	$\overline{R}$ *	$\overline{I}_T$ (MJ/day·m²)	$\overline{R}_b$	$\overline{R}$ *	$\overline{I}_T$ (MJ/day·m²)
Jan	-21.3	0.267	20.76	15.83	11.9	19.30	14.67	11.0
Feb	-13.3	0.328	6.129	4.80	15.4	5.967	4.61	14.8
Mar	-2.82	0.240	2.595	2.56	25.1	2.776	2.60	25.4
Apr	9.40	0.261	1.144	1.46	24.5	1.454	1.59	26.8
May	18.80	0.336	0.626	1.10	22.6	0.974	1.26	25.8
Jun	23.30	0.349	0.453	0.74	16.6	0.810	1.01	22.6
Jul	21.50	0.385	0.517	0.80	15.5	0.872	1.06	20.5
Aug	13.80	0.404	0.864	1.02	14.4	1.198	1.26	17.9
Sep	2.22	0.446	1.819	1.80	14.3	2.072	1.89	15.0
Oct	-9.60	0.449	4.370	3.21	11.8	4.374	3.16	11.6
Nov	-19.1	0.398	13.04	8.60	10.2	12.26	8.06	9.5
Dec	-23.3	0.408	45.68	27.80	6.4	40.72	24.80	5.7

δ = Declination of 15th day of each month

$\overline{D}/\overline{I}_H$ = Ratio of diffuse radiation to total radiation (daily average)

* Snow assumed on ground from September through May

We can evaluate the operating threshold of the solar collector, i.e., the minimum solar insolation rate necessary for the collection of solar energy for specified conditions, by use of Moan's [2] relationship for the ETSC efficiency. This efficiency equation is written:

$$\eta = \frac{A_c}{A_p} F_R \left[(\tau\alpha) \, \Phi \, \frac{I_{eff}}{I_T} - \Pi \, U_L \left(\frac{T_i - T_a}{I_T} \right) \right], \tag{4}$$

where:

η = the solar collector efficiency

A_c/A_p = ratio of absorber tube cross-sectional area to the total effective absorption area of the collector module

F_R = collector heat removal factor

Table 3. Values of the average hourly solar radiation on a tilted surface (I_T) for Fairbanks (latitude - 64° 49' N) (collector tilt = latitude).

Month	Hours Before or After Noon						Average Daily Total T_T
	± 1/2 hr I_T=	± 3/2 hr I_T=	± 5/2 hr I_T=	± 7/2 hr I_T=	± 9/2 hr I_T=	± 11/2 hr I_T=	
Jan	2.23	1.87	1.05	0.35	--	--	11.0
Feb	2.43	2.14	1.61	1.02	0.20	--	14.8
Mar	3.64	3.26	2.68	1.89	1.03	0.20	25.4
Apr	3.58	3.27	2.78	2.16	1.32	0.50	26.8
May	2.91	2.78	2.52	1.92	1.47	0.82	25.8
Jun	2.27	2.16	1.90	1.56	1.25	0.80	22.6
Jul	2.13	1.98	1.81	1.47	1.14	0.73	20.5
Aug	2.22	2.06	1.80	1.42	0.95	0.47	17.9
Sep	2.08	1.89	1.56	1.17	0.66	0.14	15.0
Oct	1.78	1.59	1.24	0.82	0.37	--	11.6
Nov	1.82	1.52	1.17	0.24	--	--	9.5
Dec	1.72	0.93	0.20	--	--	--	5.7

Units of I_T (hourly) and T_T (daily) are $MJ/m^2 \cdot \Delta t$
(where Δt = hour or day)

$(\tau\alpha)$ = cover transmissivity-absorber absorptivity product

Φ = incident angle modifier

I_{eff}/I_t = radiation enhancement term (due to reflections)

U_L = collector heat loss coefficient

T_i = inlet temperature to collector

T_a = ambient (outside) temperature

I_T = solar insolation on a tilted surface.

For the Owens-Illinois ETSC, Moan [2] reports values of:

$A_c/A_p = 0.42$; $F_R = 0.88$; $(\tau\alpha) = (0.92)(0.85) = 0.782$;

$\Phi(I_{eff}/I_T) = 2.01$; $U_L = 0.854\ W/m^2 \cdot {}^\circ C$ $(0.15\ Btu/hr \cdot ft^2)$

for a mass flow rate of air of $\dot{m} = 39\ kg/hr \cdot m^2$ of collector area (8 lb/hr$\cdot$ft^2). Thus from equation (4), we obtain:

$$\eta = 0.581 - 0.9916 \frac{W}{m^2 \cdot {}^\circ C} \left(\frac{T_i - T_a}{I_T}\right) . \tag{4'}$$

The ETSC operating threshold solar insolation rate (I_T*) is normally obtained from equation (4') by assuming an efficiency of $\eta = 0$. So that, for ambient conditions of -10°C and a collector inlet temperature of 60°C, we would obtain $I_T* = 120\ W/m^2$. However, such a simplification is not realistic because we have not accounted for the usage of electric power to move the air through the collector.

To account for this power usage, we can note that:

$$\eta = \frac{Q_u}{A_p I_T} , \tag{5}$$

where Q_u is the useful energy collected by the solar collector and A_p is gross area of the collector. In addition, we define P as the ratio of the electrical power utilized to operate the ETSC array to the total useful energy collected. Thus:

$$P = \frac{\text{blower power}}{Q_u} . \tag{6}$$

Moan [2] has reported a blower power of 41 watts (for a blower efficiency of 20 percent) for a 72 tube module ($A_p = 7.7\ m^2$) and a mass flow rate of 39 kg/hr$\cdot$m^2 of collector area. Combining these numbers with equations (4'), (5), and (6), we obtain:

$$I_T^* = \{1.71\ (T_i - T_a)/{}^\circ C + 9.17/P\}\ W/m^2 . \tag{7}$$

The choice of a maximum P, i.e., the maximum ratio of electrical energy consumed in collecting useful solar energy could be set at 100 percent, as an absolute threshold, but this is unrealistic and in addition does not account for the production efficiency of electrical energy. If we assume an efficiency of 25 percent for electrical generation and a 75 percent efficiency for direct conversion of fuel to heat, then the

maximum P which can be considered is now approximately 33 percent. On a realistic basis, therefore, we may require an operating threshold such that Q_u is twice that of the electrical power used. In this case P ≈ 15 percent and equation (7) becomes:

$$I_T^* = \{1.71\ (T_i - T_a)/°C + 61.0\}\ W/m^2. \qquad (7')$$

Table 4 lists a variety of values for I_T^* for the different operating conditions.

Table 4. Operating thresholds (I_T^*) for different operating conditions.

T_i-T_a (°C)	P = Ratio of Electric Power Usage/Solar Energy Collected					
	1%	5%	10%	15%	20%	33%
20°C	3.42	0.78	0.45	0.34	0.29	0.22
30°C	3.48	0.84	0.51	0.40	0.35	0.28
40°C	3.55	0.91	0.58	0.47	0.41	0.35
50°C	3.61	0.97	0.64	0.53	0.47	0.41
60°C	3.67	1.03	0.70	0.59	0.53	0.47
70°C	3.73	1.09	0.76	0.65	0.60	0.53
80°C	3.79	1.15	0.82	0.71	0.66	0.59

Units of I_T^* are $MJ/m^2 \cdot hr$

CONCLUSIONS

A comparison of Tables 3 and 4 indicates substantial collection of solar radiation for a variety of operating conditions, even in winter months. Table 5 shows the resulting calculations for Fairbanks and operating conditions of T_i = 60°C (140°F) and an operating threshold defined by P = 5 percent.

It becomes evident that the use of evacuated tube solar air heating collectors is a viable possibility for applications in high latitude and cold climates.

Table 5. Useful solar energy collected in Fairbanks (latitude = 64° 49' N), tilt = latitude, T_i = 60°C, P = 5% (threshold).

Month	Average $T_i - T_a$ (°C)	Average Hours per Day Operation Hours/Day	Approximate Total Hours of Operation Hours/Month	Approximate Q_u MJ/m²·month	Approximate Monthly Power Usage MJ/m²·month
January	81.7	4	124	102	2.4
February	77.6	6	168	138	3.2
March	70.6	8	248	356	4.8
April	59.9	10	300	393	5.8
May	49.7	10	310	360	5.9
June	43.1	10	300	283	5.8
July	42.3	10	310	264	5.9
August	45.4	9	279	247	5.3
September	51.6	8	240	181	3.5
October	61.3	6	186	114	2.2
November	74.7	5	150	94	2.9
December	81.4	2	62	43	1.2
Total				2,575	

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ICE FOG SUPPRESSION IN ARCTIC COMMUNITIES

By Terry T. McFadden(1)

INTRODUCTION

Ice fog is a problem that has plagued settlements since the beginning of community design in the North. It is peculiar to locations where temperatures will drop to -30°C or lower and ready sources of water or water vapor exist. Ice fog differs from normal fog in that its particles are very small ice crystals (8 to 35 μm in diameter) rather than water droplets. Because of their small size the ice crystals are very slow to settle out, and they remain suspended in the air for a long time. Precipitation rates for ice fog are on the order of 5×10^{-4} to 5×10^{-6} g m^{-2} s^{-1} (Benson 1970). The particles have been termed "droxtals" by some becausc they exhibit many of the characteristcs of crystals and yet some of the characteristics of amorphous frozen droplets. Nevertheless, they are true ice crystals though their edges and faces are rather difficult to distinguish. Once ice fog crystals have been formed they restrict visibility until they sublimate or eventually settle out. Increased air temperature will increase the moisture-holding capacity of the air and allow sublimation of the fog until a new equilibrium saturation is achieved. The arrival of a cloud cover which inhibits radiation loss to the outer atmosphere usually marks an end to an ice fog period. Arrival of winds, accompanying a weather front which breaks up the stable air layers and disperses the fog, will also bring the ice fog session to an end.

The environmental conditions that accompany severe ice fog are those of low visibility and restricted mobility for both people and vehicles. In addition, some reports of emotional stress and other health problems have been linked to ice fog. Its presence after several days becomes a severe depressant to some people, resulting in increased absenteeism from work and school. The visibility inhibiting effects of ice fog, combined with the low temperatures necessary for its formation, cause worker efficiency to drop to a fraction of its normal level. The particles which often constitute the nuclei of the ice fog crystal (Ohtake 1969) are also feared to cause respiratory problems when inhaled. The concentration of pollution products absorbed on the ice particle surfaces is the subject of some concern (Benson 1970).

Very stable air is necessary for the onset of an ice fog period which might inundate a community. This type of stable air condition is the result of temperature inversions in which the layers of air near the ground become much colder than those above. The heavier cold air is therefore very stable and resists convective mixing. Ice fog develops in this stable air and visibility quickly drops. It has been

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the frequent experience of aviators in Alaska to find extremely good visibility aloft during ice fog periods because of the presence of a high pressure area with resultant clear skies. These weather conditions, however, encourage net radiation heat loss from the air layers near the ground and result in the development of ice fog. When a pilot arrives at his destination he finds that, although visibility in every direction may be several hundred miles, he cannot see the airport 200 or 300 m under the fog and cannot land his aircraft.

The physical restrictions on travel and movement within a community inundated with ice fog, as well as the emotional and physical health problems that probably accompany it, are therefore the primary concerns that justify ice fog suppression. Suppression of ice fog can be accomplished by several techniques, but each must be considered in regard to its applicability and cost.

ICE FOG SOURCES

Before a rational program of suppression can be attempted, one must consider the sources of ice fog. They can be classified into three general categories:

1. Combustion Products. The products of combustion from virtually all fuels used by man contain water vapor, usually in large amounts. Exhausts from internal combustion engines used in machinery, automobiles and aircraft all contain a great deal of water vapor. (A gasoline engine will generate one liter of water for every liter of fuel used by the engine.) Combustion in furnaces and home heating systems provides another large and diffuse source of ice fog to the community. The products of combustion from coal, wood, and heating oil all contain water vapor that is a prime source of ice fog.

2. Open Water. Open water sources in and around a community are prime sources of ice fog. Powerplant cooling ponds, sewage outfalls, and warm springs are all water vapor sources. Evaporation from such sources can be observed to occur in a very predictable format. Water vapor entering the air above the surface of the pond reduces the density of the air, both by raising its temperature and lowering its mass density. The lighter air rises in a characteristic plume above the pond. It is then replaced by fresh unsaturated air entering around the sides of the pond as the process continues. The plume will be seen to rise for several tens of meters while moving with the prevailing drift to inundate everything in its path.

3. Minor Sources. Minor sources of ice fog include such things as exhaust ventilation from buildings and mine shafts. Most of these are of small volume and can be ignored.

HEAT BUDGET DURING ICE FOG

Heat transfer during ice fog is an extremely complex problem and is the subject of many papers. Radiation, evaporation, and convection all occur.

Experiments in Alaska during ice fog have shown that the predominant source of heat transfer from cooling ponds is convection, producing about 48% of the total heat loss from the pond while evaporation and radiation each produce about 25 to 28%. These proportions change dramatically with wind and with changes in the temperature difference between the outside air and the pond surface (Fig. 1).

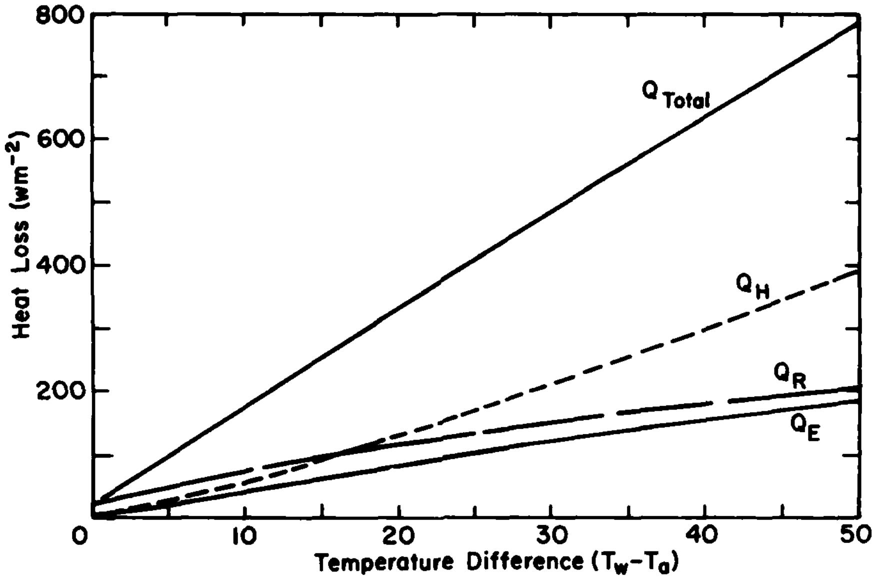


Figure 1. Heat transfer components of ice fog.

Equations based on the work of Rimshaw and Donchenko (1958) have been developed for evaporative and convective heat loss. They show good agreement with experimental data during ice fog periods (McFadden 1976). For evaporation the equation takes a form of:

$$Q_E = [4.84 + 0.21\ (T_w - T_a)](e_w - e_a) \qquad (\mathrm{W\ m^{-2}}) \qquad (1)$$

where:

Q_E = evaporative heat loss ($\mathrm{W\ m^{-2}}$)
T_w = temperature of water surface (°C)
T_a = temperature of air (°C)
e_w = saturation vapor pressure at water temperature (mb)
e_a = saturation vapor pressure (mb) at air temperature*.

* It must be remembered that this approximation will only be usable at temperatures well below 0°C. For water temperatures of 10°C, the maximum error introduced by this approximation is as shown below.

Temp. of air (T_a)	Vapor pressure at water temp. (e_w)	Vapor pressure at air temp. (e_a)	e_w-e_a	e_w-e_{ai}	Maximum error (%)
-20	12.3	1.25	11.2	12.3	10
-30	12.3	0.51	11.8	12.3	4
-40	12.3	0.19	12.1	12.3	1.5

Maximum errors will be produced if the air entering the pond area is completely dry (e_{ai}=0) and air leaving the pond is saturated (relative humidity = 100%). If the air entering is close to saturation, the error approaches zero.

For convective heat loss, the following equation can be used:

$$Q_H = [3.9 + 0.17\ (T_w - T_a) + 1.9U](T_w - T_a) \quad (W\ m^{-2}) \qquad (2)$$

where:

U = wind velocity at 2 m above the ground ($m\ s^{-1}$).

The constants of both the above equations have been determined by regression analysis using data taken during ice fog periods in Alaska. The convection equation shows good agreement with a theoretical expression developed from Kay's work (1966) up to and including wind speeds of 2 $m\ s^{-1}$. For periods of wind speed of less than 2 $m\ s^{-1}$ a theoretical equation derived by Canadians Rotem and Claassen (1969) shows good agreement with the relationships of eq 2. Comparing the ratio of convective to evaporative heat transfer from eq 1 and 2 (Bowen's ratio) as a further check gives agreement well within the scatter of the data (McFadden 1976).

Radiation heat transfer from the ice fog cloud to the surface can be expressed by a formula of the form:

$$Q_R = [0.814 + 0.034\ N \exp(-0.0584Z)] + e_a\ [0.0054$$

$$- 0.000181\ N \exp(-0.06Z)]\ \sigma\ T_A^4 \qquad (W\ m^{-2}), \qquad (3)$$

where:

Q_R = radiation heat transfer ($W\ m^{-2}$)
e_a = saturation vapor pressure (mb) of the air at 2 m
N = cloud cover in tenths
σ = the Stephan-Boltzmann constant
T_A = absolute temperature of the air, °C
Z = height of clouds in thousands of meters above the water surface.

This equation agrees with data taken during ice fog conditions with a standard error of 10.7%. Using these equations one can calculate the heat transfer characteristics from open water during the ice fog period. Heat transfer from plumes of ice fog or individual crystals is more complex.

SUPPRESSION TECHNIQUES

Several questions must be answered before considering ice fog suppression. Since many suppression techniques are very costly, the economic impact of ice fog on the community must be assessed. This is difficult and must be done individually for each community. The costs of loss of trade revenue, extra transportation problems, lost productivity, equipment damage in accidents, loss of human life in fog-caused accidents,

etc., must be assessed and an economic value placed on each. These economic analyses are subject to wide variations from community to community.

Many ice fog suppression techniques are not well developed and their effectiveness has not been established completely. Some give partial suppression at a lower cost while others give total suppression at a much higher cost. The individual needs of each community must be assessed.

Internal Combustion Engine Exhaust:

Improved design has made automobiles more capable of running in cold weather, so that ice fog problems have grown proportionately. This fact suggests that one of the best ways of controlling ice fog is to restrict the use of automobiles during ice fog periods. Economically it is very expensive to run automobiles during ice fog periods; therefore, their use decreases automatically. However, this alone is not enough to improve conditions very much. Suppression of automobile exhausts can be accomplished in several ways:

1. Routing of major thoroughfares for vehicles away from ice fog sensitive areas will help control ice fog in such areas.

2. Restricting the practice of idling vehicles during ice fog periods will also provide some control, although this seems to be very difficult to enforce in Alaska.

3. Eliminating ice fog at the source by cooling the vehicle exhaust can also be effective. When the exhaust is cooled below its condensation temperature, the resulting condensate separates from the gas and is deposited on the road where it almost instantly freezes and is no longer a fog problem. This techniques has shown a good deal of promise (Tedrow 1969, Coutts and Turner 1978).

As a vehicle exhaust enters extremely cold air, the very rapid cooling rate of the water vapor droplet results in two phenomena: first, rapid cooling results in a very small ice particle that is very slow to precipitate from the air. Second, the combustion products associated with exhaust are adsorbed onto the ice particles, resulting in contaminated ice fog. Conventional heat transfer techniques have been found to be partially successful in the condensation of vehicle exhaust to remove a large percentage of the ice fog emitted by the vehicles (Tedrow 1969, Coutts and Turner 1978).

Airports

Aircraft engine exhausts present a very difficult problem, since the configuration of an aircraft makes it very difficult to do any type of exhaust condensation without hampering its performance. It may

be more practical to locate airports far enough from the communities so that ice fogs developed by the aircraft and the community are not a problem to each other.

Combustion products from buildings or vehicles in the vicinity of an airport become a problem because the airport develops its own local ice fog cloud, which reduces visibility and restricts aircraft operations. Restrictions allowing only electric heat or carefully dehydrated exhaust from combustion heat will eliminate most of the ice fog caused by local buildings around the airport.

Ice fog from the aircraft themselves, however, is a more difficult problem, the solution of which has not met with much success. Helicopters have been used to hover at the inversion level where they produce a downdraft of warm, fog-free air that is directed into the runway areas. This has met with some limited success; however, the addition of the exhaust from the large helicopter engines to the downdraft creates additional ice fog wich tends to override much of the beneficial value. It has been proposed to use fans along the runway to dissipate ice fog, but to the best of the author's knowledge, this has never been tried.

Open Water

Use of powerplant cooling ponds in the vicinity of an arctic community should be curtailed during ice fog events, and cooling necessary for the power plant operation should be accomplished by other means. Dry cooling towers have recently shown promise and are commercially available in the U.S.A. Drawing cooling water from the groundwater aquifer and reinjecting it after one pass is another method that can be utilized in some area. However, injection well clogging problems and contamination of the aquifer must be carefully avoided, and regulations may prohibit reinjection in areas where the aquifer is used as a water source. Both dry cooling tower and reinjection techniques, moreover, are expensive and technically difficult.

Suppression of fog from cooling ponds has been tried by growing a thick ice cover which is slowly melted to provide cooling during ice fog weather. Such a procedure is technically feasible but has proven not only difficult to control but also expensive to set up (McFadden 1973).

Good results in partial ice fog suppression can be obtained with the use of thin chemical films for suppression of evaporation. Films of hexadecanol and octadecanol, when used in combination with a floating reinforcing grid, have been shown to be effective in reducing up to 80% of the fog released by the pond (as can be seen in Figs. 2-4). The floating grids in Figure 3 are necessary to maintain continuity of the film across the pond area and to protect the film from tearing due to stresses caused by air movement and water currents. A floating grid fabricated from 3-m-diameter polyethylene pipe was found to be inexpensive and effective for maintaining film integrity. Films must be reestablished regularly be addition of chemicals since they are highly biodegradable, being subject to attack by common water bacteria such as flavobacterium and pseudomonas. Chemical costs are very reasonable and application time is of minor consequence.

Figure 2. Ice fog before chemical film application.

Figure 3. Ice fog during chemical film application.

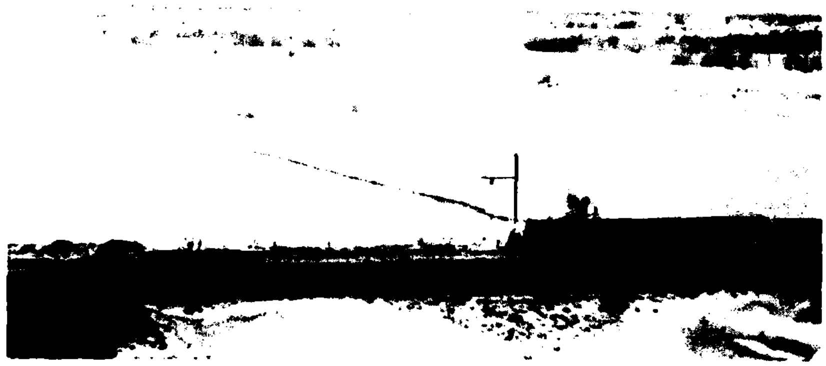


Figure 4. Ice fog after chemical film application.

Floating plastic sheets or other covers which tend to insulate the pond are not practical since they interfere with the pond's purpose, i.e. to dissipate heat.

Other sources of warm water should be discharged downwind from the community. Sources such as outfalls from sewage treatment plants or industrial wastewater cannot be reinjected into the groundwater aquifer. They must be treated and disposed of at a location where they will not form fog which would affect the community.

Very low grade heat such as that associated with powerplant cooling water (water temperatures of 10° to 15°C) has always been cheaper to discharge to the atmosphere than to use. However, the Alaskan Projects Office of the U.S. Army Cold Regions Research and Engineering Laboratory, in cooperation with the University of Alaska Horticultural Department, is currently experimenting with a means of using this heat to enhance agriculture by soil heating grids (Fig. 5). This technique of dissipating waste heat to the soil through underground pipes enables agriculturists to extend the growing season by several weeks. The warmer soil temperature produces greater yields and makes many crops possible that cannot mature in a colder soil or during a shorter grownning season. Results thus far have been excellent as can be seen in Figure 6, which shows plants in the heated plot and in the unheated control plot in July 1978. Two crops of several agricultural varieties were harvested in the 1978 growing season. Although still in its infancy, this technique holds a great deal of promise for the future.

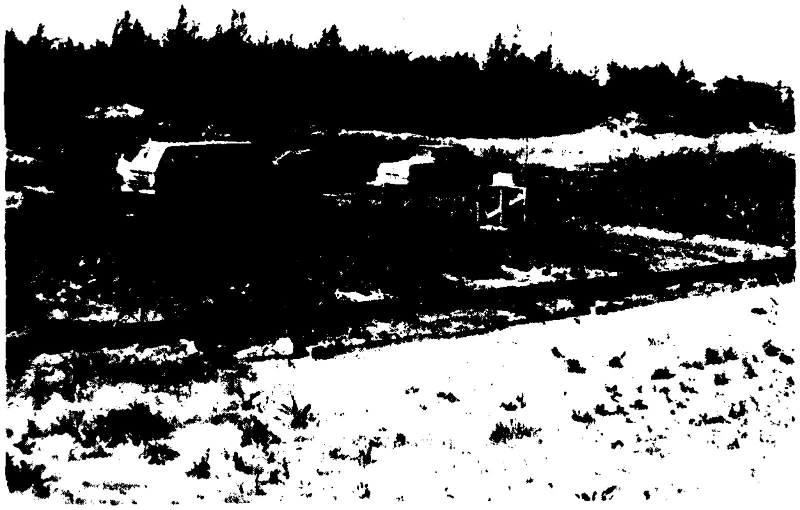
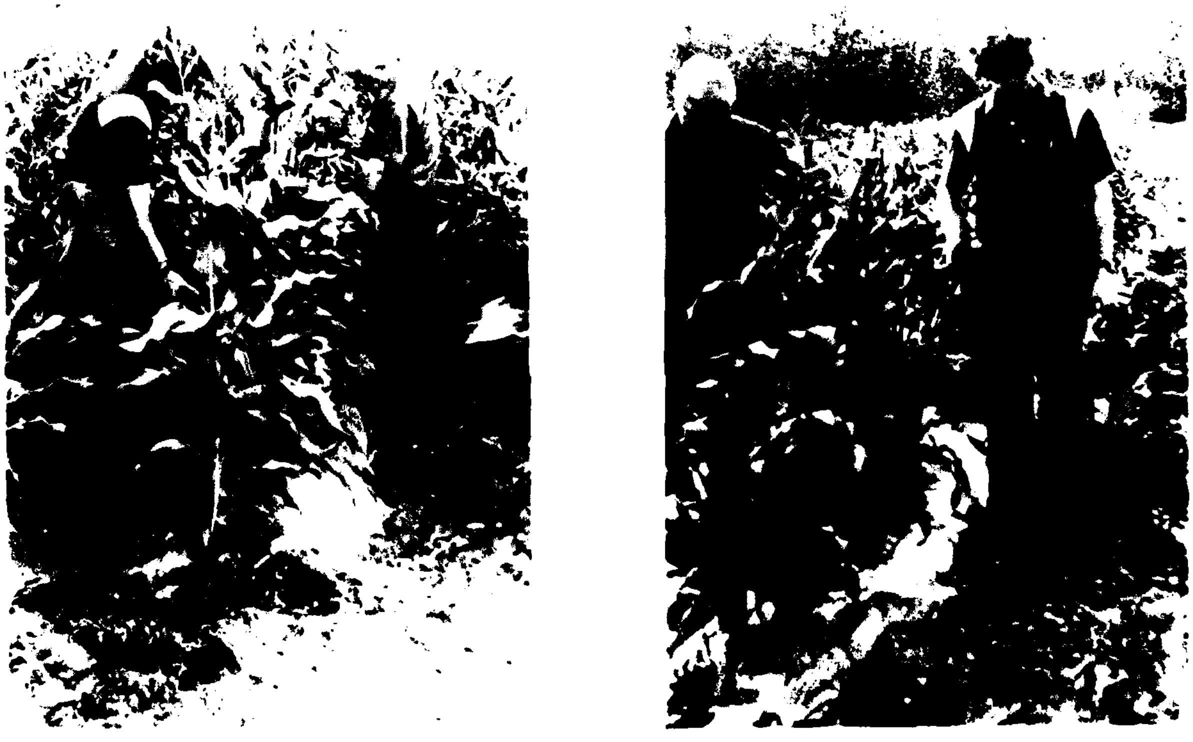


Figure 5. Waste heat recovery experimental plot.



a. Heated. b. Control.

Figure 6. Corn growth in heated and control plots.

Energy Conservation

Today's energy situation requires that all possible use should be made of any heat contained in warm water sources. Heat pump technology makes it practical to extract usable heat from such things as sewage lagoons and powerplant cooling ponds. At the Fairbanks, Alaska, sewage treatment plant, heat pumps provide the plant's entire heating requirements by removing heat from the treated sewage effluent before it is discharged into the Tanana River. This is a double saving: first, it saves on the cost of fuel for heating the facility (which amounts to several thousands of dollars a year), and second it saves on the cost of ice fog suppression since the heat removed does not then cause evaporation which produces ice fog.

Other Considerations

Before a community program for ice fog suppression can be initiated, the normal directions of air drift during ice fog periods must be determined. Prevailing winds do not necessarily bear any relationship to the air drift during ice fog periods, which occur during periods of extremely stable air. The predominant air movement at these times is gravity induced, and usual directions of this air movement are generally easily determined in two or three seasons of observation.

Electric heat has been suggested as the answer to ice fog generation from homes in the community. It certainly does eliminate ice fog from individual home furnaces; however, the total ice fog problem can be intensified unless the powerplant is hydroelectric or far removed from the community. Power plant efficiencies are usually lower than those of individual home heating plants and, if coal is used in the powerplant, it produces more water vapor per calorie of output than does oil which is commonly used in the home heating plant.

Finally, the degree of ice fog suppression necessary for each particular source must be determined. For example, it is relatively inexpensive to eliminate the first 50% to 70% of the fog generated by a cooling pond. However, the next 20% becomes more difficult and expensive and the final 10% is extremely expensive. Elimination of the first 60% means that the fog will inundate a smaller area and will be a problem for fewer days of the year (Fig. 7). Often this is sufficient. An economic tradeoff between the degree of ice fog suppression necessary and the costs involved must be determined.

CONCLUSIONS

1. Proper planning can eliminate much of the ice fog problem in new communities or in new additions to established ones. Poor planning, however, can create problems that are very expensive to correct and can have a devastating effect on the community during the very cold period of the year.

2. Established communities can do much to relieve the ice fog problem with partial suppression techniques such as chemical films and exhaust condensation. Regulations prohibiting ice fog producing

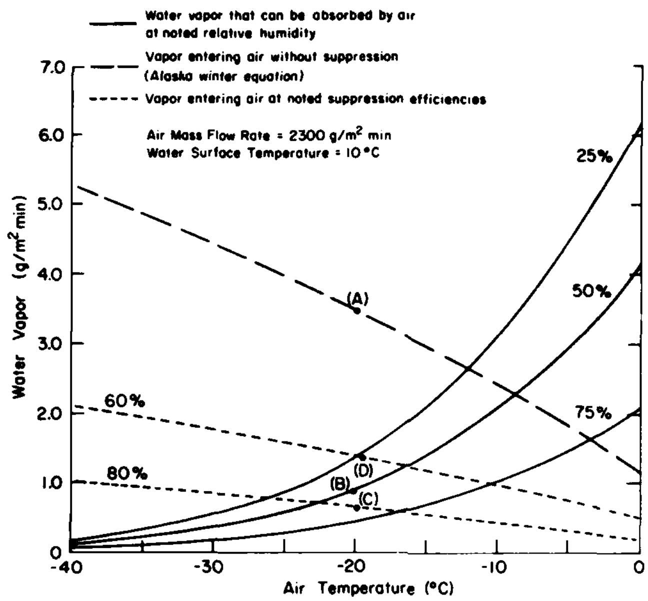


Figure 7. Ice fog vs temperature.

activities can be of considerable help during cold weather if they can be enforced.

3. Energy conservation can do much to reduce ice fog generated by the wasting of low grade heat. The higher cost of energy and recent advances in technology have changed the economics of many existing systems. These should be reevaluated to determine whether or not use of the low grade heat is now economically attractive.

4. The costs of ice fog must be determined and balanced against the cost of suppression. Often partial suppression will be cost-effective while total suppression will not.

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ESTIMATING WASHLOAD PRODUCED BY SNOWMELT EROSION ON SMALL WATERSHEDS

By Oswald Rendon-Herrero[1]

INTRODUCTION

Soil erosion of upland areas is commonly associated with surface runoff derived from rainfall. During a particular rainfall event in a drainage basin, the usual stages in erosion include wetting, softening, detachment, and overland transport of fine particulate matter to rills, gullies, and streams; raindrop impact plays a major role in the softening and detachment stages. In a basin's streams, the lateral inflow of upland-eroded sediments is known as the washload; when combined with fine sediments eroded from the stream's bank and channelbed, the material is known as the suspended load. More than 95 percent of the suspended load in most natural streams, however, is washload. Soil erosion by rainfall runoff has been studied extensively since the 1930's. From those studies, methods have been developed for quantifying various rainfall effects on soil erosion, and for estimating the relative amounts of soil eroded in a watershed.

SURFACE RUNOFF EROSION

In some cold regions and other high-latitude areas of the world, however, soil erosion is induced by surface runoff derived from melting snowpack, i.e., snowmelt. Absent during the snowmelt soil erosion process are the softening and detachment effects commonly associated with raindrop impact.

A current search of the literature for methods that can be used for estimating washload from snowmelt erosion indicates that little information is available on the subject. Commonly accepted definitions pertaining to cold regions hydrology are given by Dingman [5], Dunne and Black [7,8], and Garstka [10].

DEVELOPMENT OF THE METHOD

A method is presented for estimating the amount of washload that is produced by "sheet" erosion from snowmelt on small gaged watersheds. (For such watersheds the method requires that suspended sediment sampling be conducted on a relatively continuous basis during individual snowmelt runoff events. Data of this type, however, are rare and usually not published.) The method is based on an extension of the unit hydrograph concept developed by L.K. Sherman around 1932 for the analysis of hydrographs [16]. Application of this method using hydrologic and meteorological data from two small monitored watersheds in Pennsylvania (Bixler Run, near Loysville, 15 mi sq (39 km^2), latitude 40°-23', longitude 77°-14'; and Elk Run, near Mainesburg, 10.2 mi sq (26.5 km^2), latitude 41°-51', longitude 77°-1') indicates that the unit graph of

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measured suspended sediment discharge is related to its concomitant unit graph of snowmelt discharge (14). (Granulometric measurements in Bixler Run disclosed that practically all of the suspended sediment in the stream derives from the lateral inflow of washload from the land surfaces; a negligible amount of suspended sediment is obtained from the stream bank and channel-bed.) All of the "local" meteorologic data necessary for snowmelt hydrograph synthesis for Bixler and Elk Run watersheds is not available. An attempt to extrapolate meteorologic data from nearby areas for correlation with hydrological characteristics was unsuccessful. For a description of Bixler and Elk Run Watersheds, the reader is referred to Rundon-Herrero [14] and the USGS [19].

It is normally expected that a unit hydrograph, for a given duration of excess runoff, will change its shape and locus if a basin's flow regime is altered. Future research is, therefore, proposed for investigating the relationship between unit sedimentgraphs of a given "duration" of snowmelt runoff excess, before and after relatively large areal man-made disturbances to the land surface are effected in a particular basin. [It currently is not feasible to determine the time (i.e., duration) over which surface runoff from snowmelt occurs. For rain, a hyetograph and concomitant hydrograph usually suffice in determining the duration. There is, however, no such thing as a hyetograph for snowmelt, and therefore, the duration cannot be determined by conventional means. A method is suggested in this paper for determining duration from a graph of solar radiation-time.]

The proposed method can be used to estimate washload from snowmelt in small non-permafrost-soil watersheds if the required hydrologic and meteorologic data are available.

The work to prepare this paper was performed during June and July 1978, after an abstract for the proposed topic had been accepted. A considerable amount of hydrologic and meteorologic data that had been requested from different sources, as well as numerous useful and timely references from countries outside of the United States, were either not available, not translated into English, or not received in time to provide a more comprehensive treatment of the paper.

SURFACE RUNOFF

When rainwater can no longer infiltrate into the ground, it can accumulate and/or flow on the surface. In the absence of anteceding conditions, such as extended periods of rainfall, overland flow can occur when the intensity of precipitation exceeds the ground infiltrating capacity of the rain. Garstka has indicated that overland runoff from snowmelt is rare because the melting of snow usually takes place at rates considerably below the infiltration capacity of soil [1]. Garstka also points out that in the few instances that overland runoff has been observed, it has occurred because of a relatively thin soil mantle, or when disappearing snow on south-facing slopes exposes

the soil to frost penetration in the late spring. According to Garstka, parctically all snowmelt runoff enters stream channels as subsurface or groundwater flow, or usually as a combination of both. He indicates that in the western United States 3 to 5 inches of water may be required to satisfy the soil moisture deficiency before any marked runoff appears in the stream channels. Similarly, Eschner, et al., found no evidence that surface runoff exists for observed snowmelt events in the Adirondacks of New York [9]. They feel that it is probable that much of the water from melting snow appears in the streams as a result of rapid shallow subsurface flow before complete saturation of the soil mantle occurs.

Baker and Mace, however, found that runoff during the spring in northern Minnesota is often in the form of overland flow [2]. This, they point out, is due to soil permeabilities which are often reduced by soil freezing or through saturation of the upper soil layer by melting snow or spring rains. Dunne and Black also found that melt-water percolation during the winter melts refroze at the soil surface and within the upper foot of soil [7]. They also reported that even on parts of the watershed not covered by concrete frost (soil with no ice-free pores), complete saturation of soils is responsible for overland flow on some areas of river basins in humid areas. This overland flow is "extremely sensitive to rapidly changing inputs of water, whether from rainfall or snowmelt." Reporting on three pastured plots in Vermont, Dunne and Black found that for the 1967 snowmelt period, runoff was dominated by overland runoff over a thin layer of concrete frost in the porous topsoil. Dingman pointed out that where frozen ground surface soil occurs, it has "important influences on hydrologic processes and the water balance, including: infiltration, and hence runoff from rain and snowmelt; movement of soil moisture, and hence evaporation, transpiration, and groundwater recharge; movement of groundwater, and hence the underground feeding of rivers" [5].

Although it has been thought that frozen ground is impermeable, some studies have shown that water can infiltrate frozen ground. Dingman reported on work by Shalabanoff of the USSR and by others, concerning the infiltration of rainwater through frozen groundsurface soil; it was reported that almost 80% of the precipitation falling between September and May, when the ground was frozen, did not run off as overland flow [5]. Dingman reported that Mosienko had concluded that, when a soil has no ice-free pores (concrete frost) it is impermeable, and if the soil has more than 50% of its pores free of ice, infiltration occurs regardless of temperature.

The reader is referred to Dingman [5] for a comprehensive discussion of hydrologic effects of frozen ground.

The available literature yields little information concerning the mode of transport of meltwater along the snowpack-soil surface and/or in exposed areas "down stream" of the snowpack during the snowmelting period. It is not known if the meltwater beneath the

snowpack travels primarily as a "sheet," in discontinuous "sheets," or in rills and gullies incised in the snowpack-soil surface interface. Bobrovitskaya, et al., in their study of the erosion process from snowmelt and rainfall, found that it is not sufficient to represent overland flow as a continuous depth of water (i.e., a "sheet") [3]. They reported on work by Lisitsyna and Markocheva, who had made the observation that flow and soil erosion occur in the form of a temporary rill network.

The mode of meltwater transport is an important consideration in the determination of the upslope area on which erosion is occurring at any given time during the snowmelt period. The consideration is further compounded by the condition of the ground surface soil, that is, the state of saturation and/or temperature.

For this study, it is crudely assumed that overland runoff occurs in discontinuous "sheets" encompassing all of the area beneath the snowpack. (Some surface runoff downslope of the snowpack is also expected to occur in areas where the snowcover has been depleted. The predominating mode of transport, however, is expected to occur in the form of rills and gullies.)

A small watershed is often defined as one where its entire area can be covered by most storm events with an approximately uniform distribution of rainfall of a given intensity. When surface runoff occurs on those watersheds as the result of excess precipitation, it is assumed, according to conventional methods of hydrography analysis, to occur on the entire basin area regardless of the duration or intensity of the rain. Snowmelt surface runoff events, on the other hand, occur in smaller and continuously changing portions of a watershed's area. In this study, the area over which snowmelt surface runoff occurs will be called the effective snowmelt surface runoff area, or simply, effective area. The effective area and its variation with time during the snowmelt period is difficult to determine, and primarily depends on snow cover, antecedent hydrologic conditions, solar radiation, albedo, air temperature, and the temperature-state of the ground surface. Dunne and Black point out that snowmelt runoff is "strongly influenced by non-uniform characteristics of snow accumulation, concrete frost, saturation of soils, and radiation as modified by topography and cover. The combination of these factors causes the area contributing quick runoff to be dynamic in the sense that it varies during and between days. These facts suggest that the 'partial-area' concept of runoff production during rainstorms in Vermont . . . <u>may be a useful conceptual framework within which to view snowmelt runoff production in the same area</u>" [7] (underlined by the writer). (Dunne and Black define the partial-area concept as the storm runoff that is generated on only a small portion of a watershed [8]). They also point out that the partial-area concept provides a "point of departure for models of catchment behavior not based on infiltration theory, for a new look at such hydrologic concepts as the unit hydrogrpah . . ." A literature review by Dingman indicated that few studies exist concerning the variable-source area mechanism [5].

Below the snow line, a snowpack has a certain area (snow cover or effective area) at the beginning of the snowmelt period which may be less than the watershed's area. (An explanation of this is given by Garstka [10]. He has shown that snow that is intercepted by tree tops is lost by evaporation; the snow that is not intercepted accumulates and is "stored" longer than is snowpack in open areas.) In some latitudes, at the end of the snowmelt period, the snow cover may disappear entirely. During the snowmelt period, therefore, the snow cover may generally vary between an area which is less than the watershed area to a value of zero. The U.S. Army Corps of Engineers has found that, because it is difficult to obtain direct observations of snow-covered area, it is usually estimated indirectly [18]. Such estimates, however, may be "considerably in error, and in the overall snowmelt runoff determination, the snow covered area may be known less precisely than snowmelt or runoff factors" [18]. Usually, direct observation of snow cover is accomplished by direct reconnaissance or photography, either from the air or from the ground.

For the Boise River above Twin Springs, Idaho (830 square miles), the U.S. Army Corps of Engineers developed a generalized snow cover depletion curve, which is reproduced in Figure 1 [18]. Figure 1 indicates that the initial snow cover for the Boise River catchment area varied from about 85 percent of the total on March 31, 1954 to about 2 percent by June 30, 1954. A similar snow cover depletion curve for the St. Louis Creek Drainage Basin, Fraser Experimental Forest, Colorado, is shown on Figure 1. The depletion curves shown in Figure 1, have a characteristic "S-shape," where the rate of snow cover depletion is low at first, high during the middle of the snowmelt period, and low again at the end.

The erosive effect of overland flow from meltwater on the soil surface is complex due to the variation in the contributing area of frozen and unfrozen ground and the snow cover and bare area; also important is whether the meltwater flows as a sheet or discontinuous sheets, is concentrated in rills and/or gullies, or a combination of all of the latter. Pearce points out that any overland flow that does occur during the early part of the snowmelt period flows over either a snow-ice surface or over frozen ground, producing little if no erosion [12]. During the last part of the snowmelt period, he indicates that most of the meltwater flows over a bed of ice in the bottom of the gullies. "Only during the last few days of the spring thaw, when only 5-10% of the ground surface still has snow cover, does meltwater finally begin to erode significant areas of the streambed and banks." Dingman, reporting on the work of Atkinson and Bay, noted that frozen soil is very susceptible to erosion in the early stage of thawing [5]. At this stage, supersaturated soil is present at the surface, and any rain that falls cannot infiltrate; thus a slurry is produced that can carry a high proportion of the thawed topsoil.

For the Lower Truckee River, Nevada, Glancy, et al., believe that during snowmelt, channel erosion dominates over sheet and rill erosion [11]. (This conclusion, they say, was based on particle-size measurements made at the gage station during a rainfall event that produced overland flow primarily and during a snowmelt event in which channel

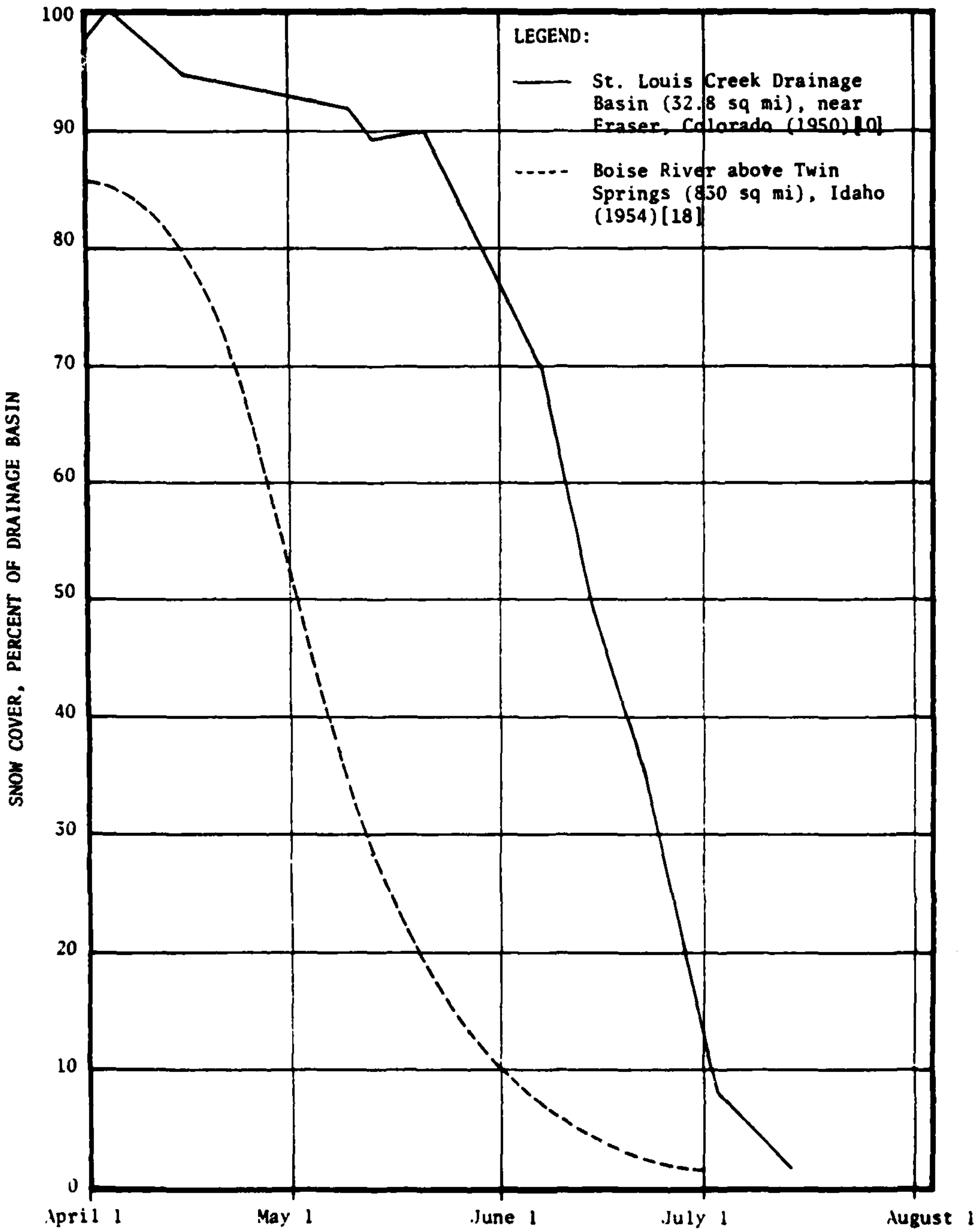

Figure 1. Snow cover depletion.

erosion predominated.) This is because main-channel erosion is believed to involve generally coarser-grained sediments than those derived from overland flow [11].

Bobrovitskaya, et al., reported on the work of Bogolubova and Karaushev, who suggested a new theoretical model of rill runoff from slopes and developed a scheme for computing soil erosion caused by rills of different sizes based on a relationship between the rate of erosion and rill magnitude [3]. Bobrovitskaya, et al., present a relationship for computing rill network soil erosion from snowmelt or rainfall on upland areas in small watersheds. The relationship accounts for the type of runoff processes, soil types and land use, and length and inclination of slopes. The general form of the relationship is as follows:

$$q = abc\ R_p^n, \tag{1}$$

where:

- q = specific sediment yield with probability of exceedance p% during runoff period (t/ha);
- R_p = runoff depth during the snowmelt or rainfall flood with the given probability of exceedance p% (mm);
- a = coefficient taking into account the influence of the land use practice of the previous year;
- b,n = parameters depending on the type of rill network on the slope, agrotechnical practice and soil types; and
- c = coefficient for the influence of slope inclination upon soil erosion [3].

Drozd, et al., point out that not all of the soil eroded in a drainage basin by snowmelt reaches the gaging station [6]. This, they point out for their study, is because discontinuous meltwater runoff, the refreezing of the thawed soil layer, and the presence of vegetation in the ravine, promote the accumulation of eroded soil in furrows and the formation of small alluvial fans on the ravine slopes.

For an excellent discussion on changes in the snowmpack and condition of the groundsurface during the progress of the snowmelt period, the reader is referred to Dunne and Black [7].

MELTING SNOWPACK

The melting of a snowpack occurs as the result of a complex heat exchange process including radiation, convection, and conduction. Radiant heat from the sun is thought to be the more significant melt factor, whereas latent heat of vaporization, released by the condensation of water vapor, and heat by conduction from the environment in contact with the snow (i.e., from the ground, rainfall, and air), are thought to be less important factors. The snowpack also loses heat as the result of radiation, sublimation, and conductivity.

Garstka states that the "interaction of the various phenomena of heat exchange make the problem of the melting of snow one of the most complex in the field of hydrology" [10].

The snowmelt in inches of water equivalent that can be produced at a point by all sources of heat can be computed by the following equation [3]:

$$M = \frac{\Sigma H}{203B} , \qquad (2)$$

where

- M = snowmelt in inches of water equivalent;
- ΣH = sum of all heat components (calories/sw. cm.);
- B = thermal quality, ratio of heat required to melt a unit weight of the snow to that of ice at 0°C; and
- 203 = constant, calories/sq. cm. required to melt 1 inch of water equivalent of ice at 0°C (80 cal./gm x 2.54 cm/in).

The primary sources of heat are:

H_{rs}, absorbed solar radiation (shortwave);

H_{rc}, net longwave (terrestrial) radiation;

H_c, convection heat transfer from the air;

H_e, latent heat of vaporization by condensation from the air;

H_g, conduction of heat from the ground; and

H_p, heat content of rain water.

Generally, conduction of heat from the ground (H_g) is a negligible melt source. The amount of heat transferred by solar radiation (H_{rs}) to the snowpack is a function of latitude, season, time of day, atmospheric conditions, forest cover and reflectivity of the snow [17].

For some situations, a less complex relationship than that of equation number 2 is available for computing snowmelt. Referring to the Fraser Experimental Forest, Garstka, et al., stated that the "indicated result of the statistical analysis of the factors causing snowmelt runoff lead to the conclusion that, for the data used in these cooperative snow investigations, the temperature factor is at least as good as, and in many cases better than a combination of other factors used in correlation analysis" [10]. They, however, point out that the "finding that temperature alone is a very important and perhaps the most important factor in snowmelt, is true for the high-altitude Rock Mountain terrain . . . It is known that high humidities, such as those which prevail in the coastal region of the Pacific Northwest, California Sierras, and in the northeastern United States, can have a very important influence upon the rate of snow melting . . ."

TABLE 1. HYDROLOGIC AND CLIMATOLOGIC DATA FOR SELECTED SNOWMELT SURFACE RUNOFF EVENTS: BIXLER AND ELK RUN WATERSHED

Watershed	Date	[d]E.R.	E.S.	t_b (time)	t_p (time)	$t_b - t_p$ (hours)	h_p (feet)	h_b (feet)	$h_p - h_b$ (feet)	Δt_B	T_w (^oF)	[a]T_a (^oF)	[b]Sunshine total (hours + minutes)	[b]Sky Cover (sunset to sunset, tenths)	[c]Solar Radiation Langleys
Bixler Run	3/10/67	.175	6.059	10:00 AM	6:30 PM	8.5	5.35	3.7	1.65	16	47	28/70/49	11:42	6	463
	3/11/67	.177	3.86	9:00 AM	6:30 PM	9.5	5.38	4.27	1.11	19	38/45	39/66/53	6:30	10	137
	3/01/66	.091	2.018	9:00 AM	2:00 PM	5.0	4.8	4.16	0.64	19	36/42	35/55/45	7:42	6	245
	3/07/63	.132	3.11	9:30 AM	4:00 PM	6.5	5.14	3.88	1.26	15	36/41	37/44/41	5:48	6	250
	3/08/63	.091	1.234	9:00 AM	4:00 PM	7.0	4.65	3.64	1.01	17	37/42	31/49/40	7:09	5	279
	3/10/63	.070	.670	8:00 AM	2:30 PM	6.5	4.33	3.49	0.84	17	38/42	32/51/42	7:51	5	382
	3/13/63	.108	3.032	1:00 PM	8:00 PM	7.0	4.87	3.59	1.28	15	40/44	36/45/40	0:00	10	290
	3/13/62	.093	1.057	12:00 Noon	8:00 PM	8.0	4.72	3.87	0.85	18	40	35/46/41	6.59	8	250
	3/14/62	.056	.333	2:00 PM	8:30 PM	6.5	4.35	3.72	0.63	17	39	35/46/41	4:51	8	204
	3/15/62	.171	5.989	12:00 Noon	6:30 PM	6.5	5.34	3.65	1.69	14	39	32/48/40	9:24	6	402
	3/16/62	.014	.140	1:00 PM	8:00 PM	7.0	3.92	3.68	0.24	14	38	31/45/38	8:14	6	377
	2/19/61	.169	7.143	12:00 Noon	5:30 PM	5.5				20	41	37/57/47	4:06	8	420
	3/28/60	.068	2.009	10:00 AM	6:00 PM	8.0	4.33	3.5	0.83	16	48	41/72/57	5:31	7	202
	2/25/58	.107	3.20	2:00 PM	7:00 PM	5.0	4.85	3.60	1.25	15	44	34/51/43	10:47	3	386
	2/08/56	.091	3.81	8:00 AM	12:30 PM	4.5	5.1	3.77	1.33	12		33/53/43	8:19	3	308
	2/09/56	.026	.404	8:00 AM	1:30 PM	5.5	4.38	3.85	0.53	12		33/52/43	2:30	10	147
	2/07/55	.078	2.278	10:00 AM	3.30 PM	5.5	4.98	3.93	1.05	16		33/41/37	5:51	6	111
Elk Run	3/13/62	.036	.786	9:00 AM	5:30 PM	8.5	1.4	1.14	0.26	16	36	35/46/41	6:59	8	250
	3/20/62	.019	.472	12:00 Noon	9:00 PM	9.0	1.13	0.78	0.35	16	37	38/52/45	5:08	8	286
	3/22/62	.083	4.755	10:00 AM	6:00 PM	8.0	1.58	1.01	0.57	18	38	38/58/48	9:43	7	408
	4/14/58	.065	1.394	10:00 AM	6:00 PM	8.0	1.39	0.90	0.49	14	34/37	42/69/56	12:15	5	451
	2/19/56	.113	4.239	11:00 AM	7:00 PM	8.0	3:15	2.35	0.80	18		31/45/38	5:29	8	351
	3/03/56	.086	3.830	11:00 AM	7:00 PM	8.0	3.2	2.27	0.93	16		39/59/49	7:55	9	285
	3/04/56	.023	.493	10:00 AM	5:30 PM	7.5	2.63	2.29	0.34	16		32/50/41	11:26	0	452

[a]Daily maximum, minimum and average air temperature at Harrisburg, Pennsylvania.

[b]Measured at Harrisburg, Pennsylvania

[c]Daily total solar radiation at State College, Pennsylvania

[d]Definition of symbols
- E.R., excess runoff, in inches per square mile of drainage basin;
- E.S., sediment mobilized by excess runoff, in tons per square mile;
- t_b, time at beginning of rise in stage;
- t_p, time at peak in stage;
- h_p, gage height at peak in stage;
- h_b, gage height at beginning of rise in stage;
- Δt_b, stage hydrograph base time;
- T_w, water temperature in degrees fahrenheit; and
- T_a, air temperature in degrees fahrenheit.

The thermodynamics of melting snowpack and related aspects are not considered to be within the scope of this study. The reader is referred to Garstka, et al. [10] and Corps of Engineers [18] for further discussion on the subject.

DATA

Hydrologic and meteorological data from Bixler and Elk Run Watersheds were compiled for selection of isolated snowmelt runoff events. (The available watershed data include water and sediment discharge, and rainfall. Information concerning solar radiation, cloud cover, and air temperature for the watershed is not available.) For detailed information about the watersheds' characteristics, type of hydrologic and climatologic instrumentation, and monitoring period, the reader is referred to Rendon-Herrero [14] and the USGS [19].

Relatively isolated snowmelt runoff events (n = 17,7) were selected from the Bixler and Elk Run data, respectively. Hydrologic and meteorologic data for the selected snowmelt runoff events are given on Table 1. The events were selected by examining and comparing the stage hydrograph chart with its concomitant rainfall chart. (Both charts are prepared using automatic recording apparatus.) The winter-spring hydrographs, for which no rain was observed or recorded prior to and during the rise in stage, were selected as probable snowmelt runoff events. (In some cases a notation appeared on the stage hydrograph indicating that the source of runoff was indeed from snowmelt. Those particular runoff events were found to follow a definite trend (see Table 2); that is, the beginning of the hydrograph's rise occurred at about 10:37 and 10:43 a.m. and peaked at 5:08 and 6:30 p.m., respectively. Normally, the rise and peak time for hydrographs that are derived from rainfall, can occur at any time during a 24-hour period.) A similar stream discharge response was reported for the St. Louis Creek drainage

TABLE 2. AVERAGE TEMPORAL CHARACTERISTICS OF SNOWMELT SURFACE RUNOFF EVENTS FOR BIXLER AND ELK RUN WATERSHEDS

Watershed	Base Time (hours)	Beginning of rise (time)	Peak (time)	Rise to Peak (hours)
Bixler Run[a]	16.00	10:37 AM	5:55 PM	6.6
Elk Run[b]	16.30	10:43 AM	6:34 PM	8.1

[a] 17 snowmelt surface runoff events

[b] 7 snowmelt surface runoff events

basin--32.8 square miles, near Fraser, Colorado--that the "daily peak of snowmelt occurred about 8 to 10 p.m. on the day of the snowmelt, and the trough of the day's melt occurred at about noon of the following day. The time of occurrence of the peaks and troughs was contingent to a considerable extent upon the distribution of melt conditions and varied widely at the exact hour of occurrence" [10]. Garstka, et al., have indicated that the shape of the snowmelt hydrograph is a "reflection of the rate of heat increase and decline during the day rather than a drainage basin characteristic, as can be ascertained from an inspection of hundreds of individual days' rises" [10]. (As noted previously for the study described by Garstka, et al., the hydrograph is generated by a combination of subsurface runoff and groundwater flow.) The latter observation about the relationship between hydrograph shape and daily variation in solar heat input suggests a method for determining a duration time for individual snowmelt surface runoff events. That is, a graph analogous to a rainfall hyetograph can be constructed for snowmelt by plotting the rate of solar radiation in langleys per hour versus time. The resulting "hyetograph" may then be subdivided into the portions during which infiltration and surface runoff were observed. The time elapsed over which surface runoff occurred on the subdivided-graph would be called the duration of snowmelt runoff excess. In this way, unit hydrographs of snowmelt could be "classified" according to duration of excess runoff, as is commonly done in conventional unit graph analysis of hydrographs. Because of a lack of local information on solar radiation, condition of cloud cover, and air temperature, it is not currently feasible to evaluate the method suggested for determining the duration of overland flow from the snowmelt runoff events selected for this study.)

The duration of the snowmelt period for Bixler and Elk Run on the average appears to be relatively short, beginning in late February and ending in late March. Most snowmelt runoff events for the period of record in Bixler Run, 1955 to 1968, occur in March (72%) and February (28%). The middle of March is the most active snowmelting period in Bixler Run.

Table 1 also lists information on air temperature, total sunshine, sky cover, and solar radiation recorded at College Park and at Harrisburg, Pennsylvania. There appears to be little correlation between the extrapolated metorologic information and the selected snowmelt runoff events. For example, there is no significant relationship between total solar radiation (extrapolated) and amount of excess runoff. This may be the case, however, because the extrapolated data may not be representative of climatologic conditions prevailing at Bixler and Elk Run Watersheds. The extrapolated data is shown in Table 1 to illustrate the type of information that would be useful for this kind of study.

ANALYSIS

The method for data reduction and analysis used to determine the excess runoff (E.R.) and associated mobilized sediment (E.S.) for

snowmelt runoff events selected for this study is fully described by Rendon-Herrero [13]. (A generalized procedure for the synthesis of a snowmelt runoff hydrograph is available for use in regions with limited hydrologic data [4]. The procedure is referred to as the Dimensionless Snowmelt Hydrograph (DSH), and depends on the availability of mean daily air temperature and streamflow data, along with representative snowpack water equivalent values for the drainage basin. Once the DSH is obtained, its ordinates provide a basis for synthesis of runoff hydrographs likely to result from arbitrary sets of assumed or historical temperature and snowpack conditions [4]. An analogous procedure for producing a dimensionless sedimentgraph has recently been developed by Williams [20].) The excess runoff and mobilized sediment are computed using the following equations [13]:

$$E.R. = \frac{2}{24} \sum_{i-1}^{2n-1} \frac{Q_{D_i}}{A(26.9)}; \qquad (3)$$

$$E.S. = \frac{2}{23} \sum_{i=1}^{2n-1} \frac{S_{D_i}}{A}, \qquad (4)$$

where:

Q_{D_i} = the direct water discharge in cubic feet per second (direct discharge is the amount in excess base and interflow discharge);

S_{D_i} = the direct sediment discharge, in tons per day;

26.9 = a factor for converting the remaining elements of equation (3), to inches per square mile;

A = the watershed area in square miles; and

i = a subscript which refers to the time at which discharge values are measured during a particular snowmelt runoff event.

Previous sections of this paper have shown that the area over which overland flow and therefore erosion occurs from snowmelt is variable, complex, and difficult to determine. During the snowmelt period, the effective area, A, to be inserted in equation number 3 and 4, is actually a quantity which may vary between an area which is less than the catchment to zero. (The latter condition, that of zero area, is incompatible with Equation 3 and 4; however, future research may show that Equation 3 and 4 may have a different form which would allow for a zero snow cover condition.) Variations in area are due to the temperature-saturation state of the ground, snow cover, ambient conditions, and mode of overland transport of the meltwater. Information of the type required to account for variation in effective area for Bixler and Elk Runs is not available. Therefore,

no attempt is made in this study to vary the area in equation number (3) and (4) to account for the variation previously mentioned. For this study, the area used in the equations is crudely assumed to be constant and equal to the watershed area.

Concerning base flow separation from total discharge (re, Q_{D_i} and S_{D_i}), Garstka, et al., have shown for St. Louis Creek near Fraser, Colorado, that in the absence of appreciable amounts of rainfall during a snowmelt period, a "train" of hydrographs will result with the trough at each succeeding rise usually being greater than the preceeding one [10]. (The bottoms of the troughs appear to occur at about noon of each day.) The snowmelt hydrographs for St. Louis Creek are, however, reported to be derived from subsurface runoff and groundwater flow. Similar snowmelt hydrograph "trains" were observed in Bixler Run during the month of March from 6/63 to 13/63, 13/62 to 18/62, and 27/60 to 30/60 for surface runoff. Garstka, et al., have suggested a method for separating a snowmelt hydrograph into the volume of a day's snowmelt appearing in the first 24-hour period, and the volume of meltwater in recession flow. Because of insufficient data from a recession analysis of snowmelt hydrographs in Bixler and Elk Run, the method of base separation used in this study is the one commonly suggested for isolated hydrographs derived from rainfall [14].

Unit sedimentgraphs from snowmelt surface runoff events occurring in Bixler and Elk Run Watersheds are shown in Figure 2 and 3, respectively. (The unit sedimentgraph in Figure 2 were obtained by arithmetically averaging graphs having the same duration of surface runoff; this is a commonly accepted procedure in the preparation of a unit hydrograph of a given duration of runoff excess. The duration time for the unit sedimentgraphs shown in Figure 2 were estimated from a correlation between hydrograph base time and duration of excess precipitation for rainfall events occurring during the period of record (1954-1969) in Bixler Run [13]. The duration time for the unit sedimentgraphs shown in Figure 3 were not estimated.) Individual unit sedimentgraph ordinates are determined thusly,

$$USO_i = \frac{S_{D_i}}{E.S.} \qquad (5)$$

where, USO_i is the individual unit sedimentgraph ordinate in units of square miles per day.

The unit sedimentgraphs shown in Figure 2, appear to plot in an analogous fashion as do unit hydrographs having different values for duration time.

Figure 4 is a plot showing the relationship between sediment mobilized and excess runoff for the selected snowmelt runoff events in Bixler and Elk Run Watersheds. The available data, however, are not sufficient for explaining the scatter of the data points in

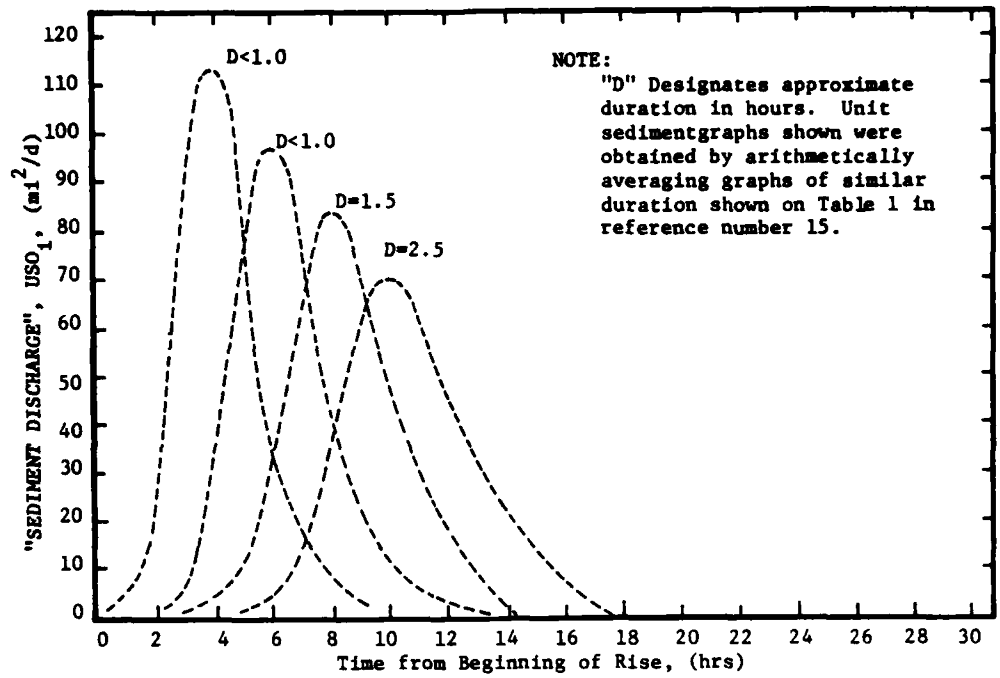

Figure 2. Unit sedimentgraphs, Bixler Run Watershed, snowmelt (15).

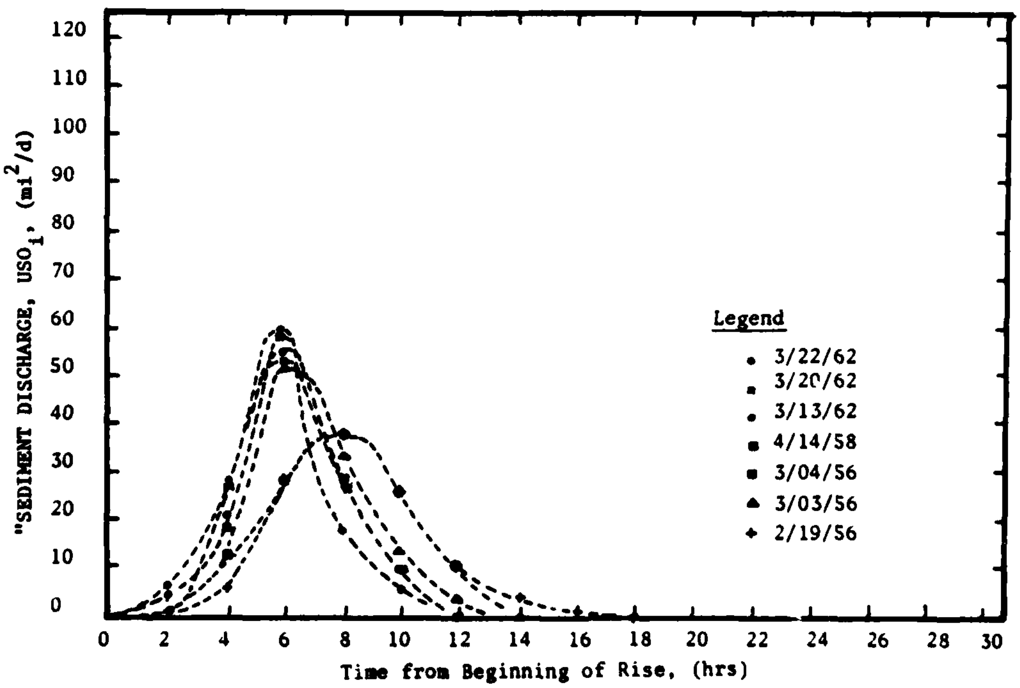

Figure 3. Unit sedimentgraphs, Elk Run Watershed, snowmelt.

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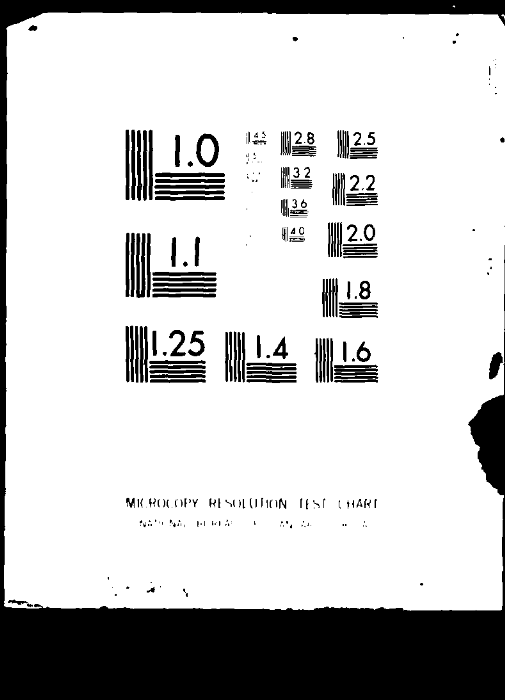

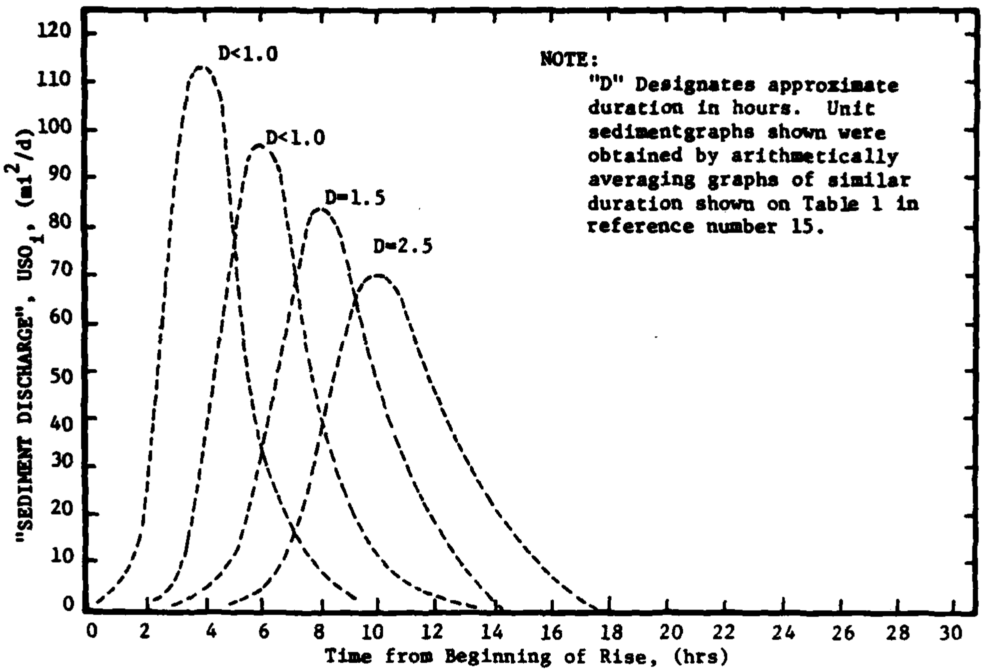

Figure 2. Unit sedimentgraphs, Bixler Run Watershed, snowmelt (15).

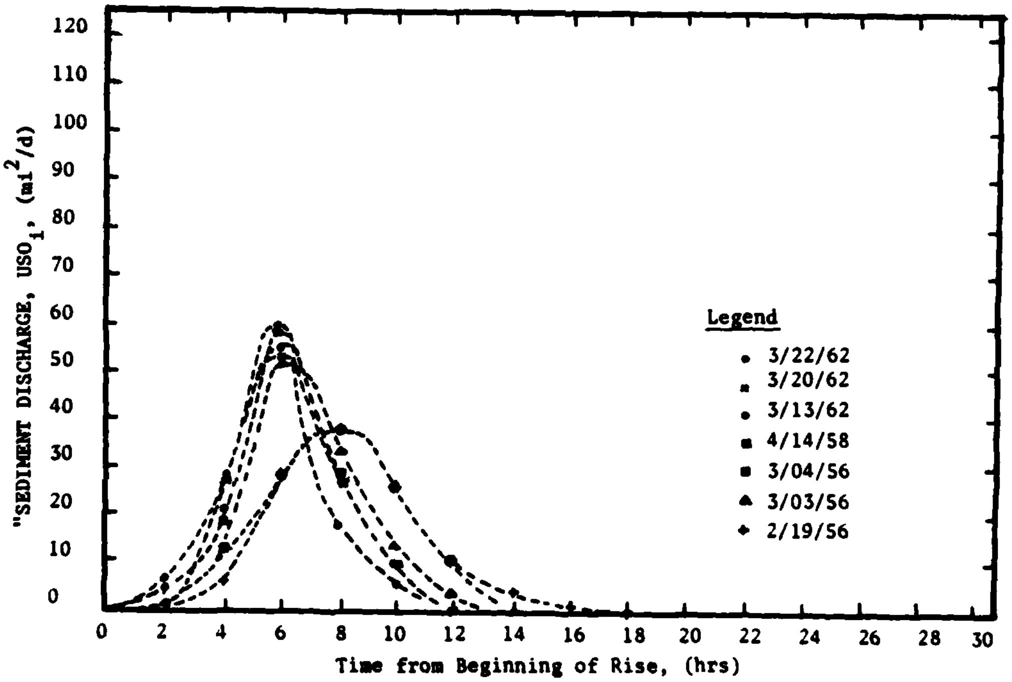

Figure 3. Unit sedimentgraphs, Elk Run Watershed, snowmelt.

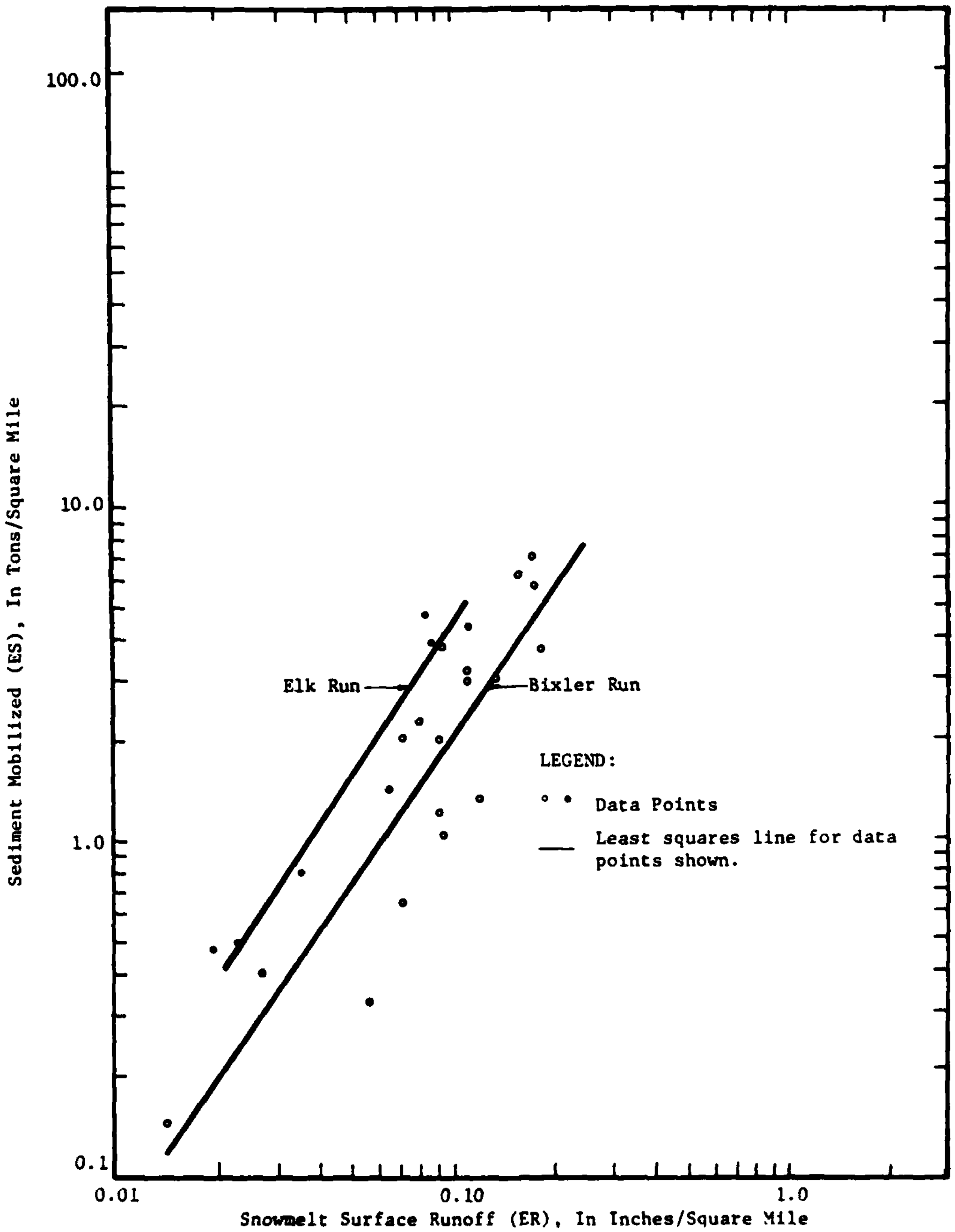

Figure 4. Sediment mobilized (ES) versus snowmelt surface runoff (ER).

Figure 4. Some of the scatter, however, may be explained by the duration time for individual events.

CONCLUSIONS

For Bixler Run Watershed, a relationship exists between washload and snowmelt surface runoff. More comprehensive hydrologic and meteorologic data from Bixler Run, however, are required for a more thorough evaluation of the relationship. The uncertainty in the relationship associated with such factors as the effective area of snowmelt erosion, duration of surface runoff, and suspended sediment sampling frequency during snowmelt events needs to be clarified.

SUGGESTED RESEARCH

The writer is of the opinion that, with improvements in the determination of snowmelt surface runoff duration time, and increased efforts to more continuously sample suspended sediment during "individual" runoff events, that the resulting unit sedimentgraphs for a given duration, will have relatively the same peak value and locus. (That is, assuming that sheet erosion occurs and is effective in eroding soil, and that conditions on the watershed remain relatively the same.) It is expected, therefore, that relatively large areal disturbances such as caused by construction operations or farming, will expose large quantities of loosened soil to surface runoff where it had not been available previously in that condition; this would result in an increase in the amount of washload "normally" mobilized. Unit sedimentgraphs of a given duration for events prior to and after such disturbances may, therefore, be significantly different from each other. The degree to which unit sedimentgraphs of a given duration are different, may well serve as a means of gaging the degree of disturbance to a watershed (or sub-watershed). Further research and experimentation, however, are required to evaluate the validity of this idea.

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SECTION II:
PRINCIPLES OF FOUNDATION BEHAVIOR, DESIGN AND IMPROVEMENT IN COLD REGION ENVIRONMENTS

FUNDAMENTALS OF THE THERMAL INTERACTION OF BUILDINGS AND STRUCTURES WITH PERMAFROST

By G.V. Porkhaev[1] and D.I. Fedorovich[2]

INTRODUCTION

Calculations of the thermal interaction of buildings and structures with permafrost include a wide range of problems in finding the temperature fields and controlling the thermal conditions of soils under different structures. Here we have to deal with very complex systems leading to the necessity of finding 2 and 3 dimensional temperature fields in freezing and thawing soils in complex conditions of heat exchange on the boundaries of the regions under investigation. Within the contemporary level of development of the theory, it is possible to bring the appropriate solutions to final numerical results only on the basis of an essential schematic representation of the heat exchange processes in the soil and in a system under consideration as a whole, which is fundamental to an engineering formulation of the problem.

At the present time engineering thermophysics has accumulated significant experience in solving similar problems and has developed sufficiently effective calculating procedures based on experimentally established laws governing the formation of temperature fields in the foundations of structures, and general physical assumptions simplifying the mathematical formulation of the problem. The following general approach is taken here.

THEORETICAL ASSUMPTIONS

On the basis of physical concepts and experimental relationships, a possibly more simple theoretical model, reflecting the principal aspects of the process in the region under investigation, and depending on the specific conditions and the nature of the problem to be solved, then is completed. It is then made more accurate as a result of an independent calculation of the influence of other factors. The individual results are linked on the basis of the heat balance relationships and experimental data.

Experience shows that this approach makes it possible to solve very complex multi-factor problems and significantly expands the possibility of using computer technology for these purposes. In addition, the accuracy of the results obtained depends not so much on the strictness of the analytical solutions as it does on the correct choice of the initial and the theoretical models and the designations of the boundary conditions of the problems.

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BASIC CALCULATING PROCEDURES

A freezing and thawing soil system may be viewed as consisting of two zones: a zone of thawed soil and a zone of frozen soil. The movement of water and water-vapor phase conversions take place in the melted zone upon a change of temperature. In the frozen zone, in addition to these processes, there are phase conversions of the pore water: bound water-ice and ice-bound water. Freezing and thawing of free water at a temperature of $t = t_f = \text{const.}$ takes place in the contact layer on the interface of the thawed and frozen zones. In both zones the transport of heat is achieved by conduction and convection.

The intensity of the process and the relative role of one process or another depend upon the composition of the soil, the moisture and temperature thereof, and also on the direction and rate of the thermoprocess (freezing and melting) and their influence on the final results. The necessity and method of taking account of them in practical calculations are determined by the natural conditions and the nature of the problem to be solved.

The experimental data available indicate that in most practical cases, with the exception of highly filtering soils, the influence of all the above mentioned factors may be accounted for fairly completely in a model of the equivalent heat conductivity of the soil, and with the internal sources of heat reflecting the latent heat of the freezing and thawing of water. According to this model, the multi-factor heat transfer in the soil is examined from the concepts of Fourier's theory of heat conductivity, the influence of complicating factors being considered in boundary conditions of the problem, and in effective values of the thermophysical characteristics of the soil.

Within the framework of this model consideration of the phase transitions of water usually reduces to the two extreme cases:

1.) all moisture in the soil freezes and thaws at a temperature $t = t_f = \text{constant}$

2.) freezing and thawing of the water takes place in the soil in a continuous temperature range.

The first case refers to the freezing and thawing of sand and very moist clay soils ($W > W_p$). In this case the problem is formulated in Stefan's traditional formulation; the ground is considered to be a 2 layered system with a mobile interface of the thawed and frozen zones on which all heat liberation of phase transitions takes place.

The temperature fields in these zones are described by the equations:

$$\lambda_M \frac{\partial^2 t_M}{\partial z^2} = C_M \frac{\partial t_M}{\partial \tau} \text{, and,} \tag{1}$$

$$\lambda_\tau \frac{\partial^2 t_\tau}{\partial z^2} = C_\tau \frac{\partial t_\tau}{\partial \tau} \qquad (2)$$

where $C_M/\partial\tau$ = Heat capacity of the frozen soil at the interface as a function of time
$C_\tau/\partial\tau$ = Heat capacity of the thawed soil at the interface as a function of time
t_τ = Temperature distribution in the thawed soil
t_M = Temperature distribution in the frozen soil
z = Depth beneath the freezing plane
λ_τ = A constant function of the subscript's heat capacity with time.

On the mobile interface of the zones with the coordinate, the heat balance is given as:

$$\lambda_\tau \frac{\partial t_\tau}{\partial z}\bigg|_{z=\xi} + \lambda_M \frac{\partial t_M}{\partial z}\bigg|_{z=\xi} = \sigma\gamma\ (W-W_H)\ \frac{d\xi}{d\tau}\ , \qquad (3)$$

where W and W_H are the total soil moisture with the unfrozen water, γ is the unit weight of the moist soil, and σ is the latent heat of the ice.

In the second case, referring to the freezing and thawing of slightly moist clay soils ($W < W_p$), where a large part of the moisture is in the bound state, the temperature field in the thawing zone is described by equation (2), and for the freezing (thawing) layer - by an equation of the form:

$$\lambda_M \frac{\partial^2 t_M}{\partial z^2} = C_{ef}(t)\ \frac{\partial t}{\partial \tau}\quad , \qquad (4)$$

where $C_{ef}(t)$ is the effective heat capacity of the soil:

$$C_{ef}(t) = C_M + \sigma\gamma\ \frac{dW_H}{d\tau}\quad . \qquad (5)$$

The form of the function $C_{ef}(\tau)$ is determined on the basis of experimental data, and on the boundary of the thawing and frozen zones, the condition $t = t_f$ is given.

In the formulations presented, both problems refer to the nonlinear class, precise analytical solutions of which are obtained only for the simplest conditions. In more general cases approximate or numerical methods of solution are used.

For a numerical solution of a nonlinear equation of the form (4), a locally unidimensional procedure usually is used. The heat conductivity

equation contains δ, the Dirac function, and it is described as:

$$\lambda_M \frac{\partial^2 t_M}{\partial z^2} = [C_M + \sigma\gamma W_H \delta(t-t_f)] \frac{dt_M}{d\tau} . \tag{6}$$

Subsequently, the smoothing method is used: the δ function is approximately replaced by δ, an inverse function of the form $\delta(t-t_f,\Delta)$, differing from zero in the interval $(t_f \pm \Delta)$, and satisfying the normalization conditions. By introducing a linear or parabolic approximation, the effective heat capacity in the interval under consideration is described by:

$$C_{ef}(t) = C_M + \sigma\gamma W_H \delta(t-t_f,\Delta), \tag{7}$$

and equation (4) is reduced to a quasilinear form.

This procedure is realized easily on a medium class digital computer and imposes relatively small limitation on the soil characteristics and boundary conditions. However, since the function $C_{ef}(t)$ where $t = t_f$ suffers interruption, the convergence of iterations in the region $t = t_f + \Delta$ is not high. In general, the convergence and stability of the solutions of nonlinear problems of the class under consideration, on the basis of the above-mentioned and other available algorithms, has as yet been proven for one dimensional self-modelling problems only. In all other cases it is necessary to perform special investigations of the asymptotic behavior of the function sought, which requires a great deal of machine time and is not always practically possible.

For solving the problem in Stefan's formulation, that is, with nonlinear boundary conditions, analog computers of the type of Luk'yanov's hydrointegrator (the use of which does not meet with significant limitations here) makes it possible to solve very diverse problems, including non-selfmodelling multidimensional problems with variable boundary conditions.

Approximate analytical solutions of problems of this type are performed by means of choosing temperature functions $t_T(z,t)$, and $t_H(z,t)$, satisfying the boundary conditions. For plane problems and these purposes, Kramif functions are used, and Besser functions are used for axisymmetric problems. These functions are substituted into Stefan's condition (3), which makes it possible to find the law governing the movement of the thawing (freezing) boundary.

SOLUTION OF THE BASIC PROBLEM

The simplest solutions are obtained in the case of an approximation of the temperature fields in freezing and thawing soils by functions of a stable distribution of temperatures in them, and a given position of the interface of the thawed and frozen zones. Here the process is viewed as quasi-stationary, which makes it possible to reduce the solution to the nonlinear problem to the solution of ordinary differential equations of the

second order. This approach (known as the principle of the successive replacement of stationary states) is the basis of most heat engineering calculations and makes it possible to obtain solutions in closed form for very complex 2 and 3 dimensional cases.

A halfspace with a cavity, the surface of which is the interface of the thawed and frozen zones, with a temperature of $t(x,y,z) = t_f = 0$, which temperatures of $t(x) = t_n = 0$ are given under the building, are usually considered in 3 dimensional temperature fields. The average annual value of the soil temperature $t(\infty) = t_0$ is taken as the limit of the thermal influence of a building. The temperature distribution in the thawed and frozen zones is assumed to be quasi-stationary.

Assuming that melting of the ice does not take place on the thawing boundary, and that a certain variable temperature $t_0^x(\tau)$ providing for the same law governing the movement of the thawing boundary is maintained on the surface of the ground beyond the limits of the building, the temperature distribution in the thawed zone is described in the form:

$$t_T(x,y,z) = t_o^x + (t_n - t_o^x)\, f(x,y,z), \tag{8}$$

where $f(x,y,z)$ is the function of the configuration of the system, t_o^x, and t_n exist as described above.

Finding the value of t_0^x from the condition that on the thawing surface with the coordinates $x = x'$; $y = y'$ and $z = z'$ the temperature $t_T(x', y', z') = 0$, formula (8) reduced to the form:

$$t_T(x,y,z) = t_n \frac{f(x',y',z') - f(x,y,z)}{f(x',y',z') - 1}\ . \tag{9}$$

The temperature distribution in the frozen zone $t_M(x,y,z)$ is found analogously.

The movement of the thawing surface with time is determined from condition (3). Describing an elementary area in the plane of symmetry of the problem, and taking account of the values found for $t_T(x,y,z)$, and $t_M(x,y,z)$; where $x = y = 0$, the integral is the expression:

$$\int_o^{\xi} \frac{f(z) - 1}{1 - \frac{\lambda_M t_o}{\lambda_T t_n}\left[1 - \frac{1}{f(z)}\right]} \frac{dz}{f'(z)} = \frac{\lambda_T t_n}{\sigma \gamma W}\ \tau, \tag{10}$$

where $f'(z)$ is a derivative of the function of the configuration of the system in the plane of symmetry.

Expression (10) makes it possible to calculate the depth of thawing of the soil $\xi(\tau)$ in the plane of symmetry at a given moment of time τ.

The temperature distribution in the soil and the position of the thawing surface under the building, at this moment of time, is determined on the basis of equation (9), assuming $f(x,y,z) = f(\xi)$. The latter equation is also the equation describing the interface of the thawed and frozen zones.

In the case of complex conditions of heat exchange on the surface, and the presence of heat liberating objects, the method of superposition of temperature fields is used. If the conditions of the problem may be expressed by sources of heat of a given intensity J_q and plane surface sources of a given J_t, the summation of the temperature fields is performed without taking account of the mutual influence of the boundary surfaces. If in addition to 1-dimensional sources (of the form J_t) located on the surface, there are sources of heat (of the form J_q) or closed surfaces with sources buried in the ground, then the problem is solved by the trial and error method for zero boundary conditions with the introduction of equivalent sources of heat into consideration.

Thus, for example, in calculating the heating influences of a hot water line with a temperature of t_{TP} on the heat conditions of soils in a foundation near a building, a temperature field is sought in the form of the sum of two functions:

$$t = t' + t'', \tag{11}$$

where t' is the temperature field in the foundation of the building in the absence of pipes with limiting temperatures t_n and t_o, t'' is the temperature field of the halfspace with the temperature $t = t_{TP}$ on the surface of the hot water line and the temperature $t = 0$ on the surface of the ground.

The value of the temperature t' is determined by equation:

$$t'_o = t_o + (t_n - t_o)\ f_1, \tag{12}$$

where f_1 is the function of the configuration of the building-ground system.

The temperature t'' is found through an auxiliary source of heat q_e, equivalent in its influence to a pipe with the temperature as shown:

$$t_{TP} = q_e\ f_2, \tag{13}$$

where f_2 is the function of the configuration of the pipe-ground system.

The value of q_e is found from the condition that the equality $t' + t'' = t_{TP}$ is fulfilled on the surface of the pipe.

CALCULATION OF CONVECTIVE HEAT TRANSFER

The applicability of the procedures presented above is limited by the condition where the convective component of heat transfer is small and is connected only with the temperature gradient of the soil. In the general case where the convective heat transfer results from factors extraneous with respect to the temperature, for example, gravitational forces, another interpretation of the phenomenon is required. In practical heat engineering calculations different modifications of the Fourier-Kirchhoff and Bukinhem-Lykov equations and also the introduction of simplifying assumptions make it possible to separate the problem into individual components which usually are used for this.

For engineering purposes there is practical interest in taking account of the influence of the filtration of underground waters and capillary film moisture transfer in the active layer of frost, in the case of incomplete core filling on the temperature conditions of soils.

The influence of filtration is taken into account in those cases where the following criterial condition is fulfilled:

$$\frac{K_f C_W \Delta H}{2\lambda_T} >> 1, \qquad (14)$$

where: K_f is the filtration factor of the thawed soil

H is the hydraulic pressure drop

C_W is the volumetric heat of the water.

For describing the temperature field in the filtration zone (t_T), the Fourier-Kirchhoff equation is used with the following simplifying assumptions:

1.) the filtration of water in the thawed zone takes place in stable conditions

2.) the temperature field in the filtration zone is quasi-stationary

3.) the heat transfer in the filtration zone in the direction of filtration is accomplished only by means of conduction, and in the direction of the normal filtration flow, only by means of heat conduction.

In the above, formulation of the problem is divided into two parts, reducing to the independent determination of the structure of the hydraulic and temperature fields, with coupling of them on the mobile thawing boundary.

For determining the temperature field in the filtration zone the Fourier-Kirchhoff equation is transformed to curvilinear coordinates of the hydraulic grid (ψ,ϕ) and on the basis of the assumptions made reduces to the form

$$\lambda_T \frac{\partial^2 t_T}{\partial \psi^2} = C_W \frac{\partial t_T}{\partial \tau} , \quad (15)$$

where ψ is potential function of the filtration rate, constant along equipotential lines, and ϕ is the function of the filtration current, constant along flow lines.

Equation (15) is analogous in its structure to the Fourier heat conductivity equation, the solutions of which are well known, which makes it possible to find the temperature fields in the filtration zone under very different conditions. Thus, for example, if we assume that the ϕ axis coincides with the melting boundaries, and the axis ψ coincides with the boundaries of the "feeding" region, then the well known formula for the temperature field in an infinite body satisfies the solution of equation (15) in the case of a value $F_O < 0.1$. Considering that in this case on the lower boundary of the filtration zone the coordinate $\psi = 0$ and coincides with the normal to the thawing boundary, then:

$$\frac{\partial t}{\partial \psi} = \frac{1}{V} \frac{\partial t}{\partial n} \quad \text{and} \quad \frac{\partial t}{\partial n} = V \frac{\partial t}{\partial \psi} ; \quad (16)$$

thus the equation of the heat balance on the mobile thawing boundary is described in the form of:

$$-\lambda_T V \left. \frac{\partial t(\psi,\phi)}{\partial \psi} \right|_{\psi=0} = \sigma \gamma W \frac{d\xi}{d\tau} . \quad (17)$$

Here, V is the rate of ground water filtration.

Since the temperature in the filtration zone varies along the coordinate ϕ, calculation of the thawing is performed for specific sections, limited by adjacent flow lines ψ.

The influence of convective heat transfer in the active layer of frost penetration (in the case of an incomplete filling of soil pores with water) may have practical significance in the case of a value of Lykov's criteria, thus:

$$Lu = \frac{a_k C}{\lambda} >> 1, \quad (18)$$

where a_k is the coefficient of capillary diffusion of water.

In a strict formulation of the problem, accounting for the moisture transfer under these conditions is performed on the basis of a joint solution of the system of equations of heat and moisture exchange. In engineering calculations, the Bukinhem-Lykov equation, on the basis of which a number of solutions in closed form can be obtained, usually is used. In this case it is found that the influence of the factors of

moisture transfer on the nature of the temperature field in the soil is manifested indirectly through the variation in the insitu moisture and the conditions of heat exchange on the interface of the media. Thus, with the accuracy necessary for practical purposes, this may be accounted for in the boundary conditions of the problem.

DESIGNATION OF BOUNDARY CONDITIONS

For solving applied problems, the conditions of heat exchange on the soil surface and in contact with buildings and structures are given in the form of boundary conditions of the I, II, and III types. In all cases, where it is possible, these conditions reduce to boundary conditions of the kind I by means of the introduction of the equivalent temperature of the external medium, therefore:

$$t_e(\tau) = t_{ext}(\tau) + \frac{1}{\alpha_k} \sum_{1}^{i} q_i(\tau) \,, \tag{19}$$

where: $t_{ext}(\tau)$ is the actual temperature of the external medium

$q_i(\tau)$ are the components of the external heat exchange

α_k is the coefficient of convective heat exchange.

The values of the functions $t_{ext}(\tau)$ and $q_i(\tau)$ are determined on the basis of experimental data with the subsequent statistical averaging in accordance with the nature of the problem being solved. In order to obtain self modelling of quasi-stationary solutions the mean integral values for the calculated period of time (or periodically established values) usually are used in the calculations.

The soil temperature beyond the limits of the thermo-influence of the building is taken as equal to its average annual value t_0. In solving problems according to the method of successive replacement of stationary dates, we must take account of the heat flow from the soil mass to the thawing (freezing) boundary. This temperature is given on the boundary of the area of the thermo-influence of the thawing (freezing) zone, the radius of which is determined on the basis of experimental data.

The heat exchange on the boundary of the thawed and frozen zones is given by Stefan's condition where $t = t_f$.

CONTROL OF THERMAL CONDITIONS

The control of the thermal conditions of soil in the foundations of buildings and structures is performed by means of constructing ventilated cellars, ventilated ducts, and other measures, making it possible to vary and regulate the conditions of the heat exchange between the soil and a structure.

Heat engineering calculations of ventilated ducts and cellars are

performed for the average annual soil temperature t_0', according to the conditions providing for the theoretical bearing capacity of frozen foundations, and taking account of the maximum permissible depths of the seasonal thawing and freezing, H_M and H_T. These calculations are based on the equation of the thermal balance of the cellar or duct and the method of determining the average annual soil temperature according to the value of the cooling pulse, hence:

$$\Omega_{ox} = t_0' \, T = \Omega_3 - \Omega_{np} \tag{20}$$

where: Ω_3 is the total of degree-hours of the equivalent air temperature in the celler during the winter period
Ω_{np} is the same for the freezing time of the seasonally thawing layer, up to its contact with the permafrost
T is a period of time equal to one year.

The value of Ω_{np} is determined from the obvious equation:

$$H_T = H_M = \sqrt{\frac{2\lambda_T \Omega_n}{\sigma\gamma W}} = \sqrt{\frac{2\lambda_M \Omega_{np}}{\sigma\gamma W}}\,, \tag{21}$$

which, taking account of equation (20), makes it possible to express t_0' in the following form:

$$t_0' = \frac{1}{T}\left(\Omega_3 + \frac{\lambda_T}{\lambda_M}\ \Omega_n\right). \tag{22}$$

where: Ω_n is the total of degree-hours of an equivalent air temperature in the cellar during the summer period.

Equation (22) makes it possible to calculate the values Ω_3 and Ω_n, and on the basis of the heat balance of the cellar or duct,to determine the necessary conditions of their ventilation.

In a cold cellar and beyond its limits, the air temperature varies periodically, which causes fluctuations of the soil temperature. The temperature field in the basement of buildings, taking account of the periodic variation in temperature on the soil surface and in the cellar $t(x,z,\tau)$, is found in the form of two functions:

$$t(x,z,\tau) = t'(z,\tau) + t''(x,z,\tau) \tag{23}$$

where: $t'(z,\tau)$ is a 1 dimensional temperature field with periodically varying temperature on the soil surface
$t''(x,z,\tau)$ is a 2 dimensional temperature field with periodically varying surface temperature on the section under the building and with constant temperature (t-0) beyond its limits.

The solution of the problem in this formulation makes it possible to obtain the extreme values of the soil temperatures under the middle and edges of the building, necessary for calculating the bearing capacity of the foundation.

There are a number of other methods of solving this type of problem which cannot be cited here.

SUMMARY

Within the framework of this report we attempted only to deal briefly with the most general approaches to the formulation and solution of applied engineering problems and to examine in this formulation the basic theoretical models and procedures used in practical heat engineering calculations. The choice of calculated procedures, the degree of their detail, and the methods of solution in each specific case are determined by the nature of the problem to be solved, the completeness of the initial data, and the physical essence of the processes under consideration.

It is important to note that in spite of substantial simplifications and model constructions of a qualitative and quantitative nature, the calculating model and procedures presented above basically reflect a real picture of the phenomenon. Also it is with a completeness and correctness acceptable for engineering purposes to make it possible to estimate the magnitude, direction and quantitative indicators of the processes for the formation of the dynamics of temperature fields, observed in nature. And the limits of the accuracy with which the initial conditions of the problems to be solved may be given.

Experience shows that in most practical cases even the simplest calculated models and procedures provided for significantly greater accuracy of the final results than the accuracy of assigning the initial calculating data. In other words, with the contemporary level of development of heat engineering calculations it is necessary to seek increased reliability and accuracy of the results obtained. Consequently, calculating data of the problem to be solved and the conditions of heat exchange on the boundaries of the region under investigation, including the insitu parameters of the soil, should account for their space-time variability and the interconnection of heat and moisture exchange processes on the surface.

The dynamic nature of frozen strata, the nonuniformity of its composition, and the dependence of the conditions of heat exchange on the climatic, hydrophysical, and other natural factors do not make it possible to count on the possibility of a significant increase in the level of reliability of assigning the initial indicators only on the basis of the deterministic approach. The most complete solution of this problem assumes the use of the apparatus of the theory of probabilities and mathematical statistics. This in turn is connected with the necessity of using a large volume of information and on the development of special methods of analyzing and generalizing it, and also specification of the direct correlation connections between the heat exchange parameters and the natural climatic and hydrophysical indicators of the section under consideration.

CONCLUSION

It is necessary to note that at the present time planning practice deals not only with the heat engineering calculation of a given structure, but also with providing for a reliable planning solution. With minimum losses, taking account of the possible change in the permafrost conditions in the territory under development, planning should include the change influenced by buildings and structures under construction. This assumes an examination of the problems of engineering thermophysics, and, in particular, choosing the principles of the utilization of permafrost as foundations from the positions of the theory of reliability and statistical dynamics which account for the stochastic nature of the initial information. A practical solution of these problems leads to the necessity of a wide utilization of computer mathematics for gathering and analyzing primary information and the development of the corresponding algorithms for the mathematical modelling of different natural and permafrost situations in the areas under development for choosing optimum types of construction.

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ANALYSIS OF FOUNDATIONS IN PERMAFROST

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INITIAL BEHAVIORAL CONCEPTS

In the newly issued construction rules and standards /1/, as before, two basic principles of using permafrost for foundations are presented: Principle I, with the frozen state of the foundation preserved and Principle II, with thawing of the ground in the process of utilization of the building or with preliminary thawing, before constructing the building. This division is due to the abrupt change in the mechanical properties of permafrost when its temperature passes through the freezing-melting point θ_0 (Fig. 1). The relationship between the strength of permafrost R_M and the temperature thereof (see Fig. 1a) may be described by the empirical formula:

$$R_M = R_{M(0)} + b\ |\theta|^{\lambda} \tag{1}$$

where $|\theta|$ is a negative soil temperature (°C), without a minus sign, $R_{M(0)}$ is the strength at $\theta = -\theta_0$, b and λ are parameters, when $\lambda \approx 0.5$.

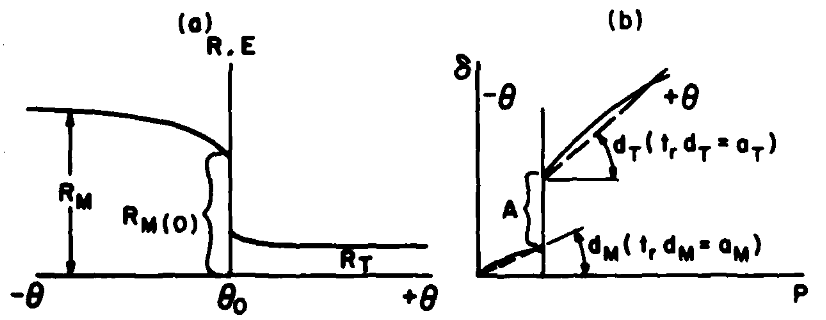

Figure 1. Variation of strength (a) and deformation (b) properties of frozen soil upon thawing.

Upon passing through the melting point the strength of a soil, as a result of the destruction of its ice-cement bonds, abruptly decreases to the value $R_{T(0)}$. With a further increase in temperature the strength barely changes, $R_T \approx R_{T(0)}$.

The change in the deformation properties of permafrost upon thawing may be illustrated by a graph of the dependence of the relative compressibility

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($\delta=\Delta h/h$) (where lateral expansion of the soil is impossible, and on the pressure p) (see Fig. 1b). The dependence of δ on the pressure is nonlinear in the general case, but in a small range of variation of p this dependence may be linearized. Then, for the frozen state:

$$\delta_M = a_M p, \tag{2}$$

where $a_M = (I - \frac{2\mu^2}{1-\mu})/E_M$ is the compacting coefficient of the frozen soil depending on its temperature; E_M and μ are the deformation modulus and Poisson coefficient of the frozen ground; p is the effective pressure.

Upon thawing, the value δ abruptly rises and the relationship between p and δ of the thawed soil has the well known form:

$$\delta_T = A + a_T\ p, \tag{3}$$

where $A = \delta_T - \delta_M$ is the thawing coefficient characterizing the abrupt increase in settling upon thawing, and a_T is the compacting coefficient of the thawed ground: $a_T >> a_M$.

The calculation of foundations is performed with respect to two limiting states:

With respect to the strength (bearing capacity), starting from the condition:

$$N \leq \phi/K_r, \tag{4}$$

where N is the calculated load on the foundation, ϕ is the bearing capacity of the foundation and K_r is the reliability coefficient; and with respect to deformations arriving from the condition:

$$\zeta \leq \zeta_{1im}, \tag{5}$$

where ζ is the deformation of the foundation determined by calculation, and ζ_{1im} is the maximum permissible value of the deformation of the structure, depending on its structural characteristics.

Since frozen soils are not highly compressible, when they are used according to Principle I (with preservation of the frozen state) a strength calculation is fundamental. Plastic frozen and highly icy soils form the exception. These soils are capable of developing perceptible settling and therefore must be calculated both with respect to strength and with respect to deformation.

In thawing soils there is usually a great amount of settling; therefore, a calculation of foundations used according to principle II must be performed with respect to deformations. A control strength calculation is performed only in the presence of shearing forces.

FOUNDATION CAPACITY OF PERENNIALLY FROZEN SOILS

Column and piling foundations are basic types of foundation used in construction according to Principle I. Beginning in the 1950s pilings became the most common type of foundation. They are usually used together with a ventilated crawl space providing for preservation of the frozen ground state of the foundation. Depending on the size of the load, the piles are single, paired, or grouped. Reinforced concrete in square or rectangular cross section or hollow circular cylindrical piles usually are used. Reinforced concrete pile shells and circular cylindrical columns measuring 800 mm and more in diameter are used for very large loads. Wooden piles are used only for unimportant buildings, tubular metal or roll steel piles are used in special cases. The piles are sunk, primarily, with the preliminary drilling of holes (greater in diameter than the cross-section of the piling); the hole is then filled with a soil slurry so that the pile will be frozen into place. In plastic frozen soils the holes are made of a smaller diameter than the cross-section of the pile and the pile is driven with a vibrator or diesel hammer; usually, it proves to be possible to drive the pile directly into plastic frozen soil without drilling lead holes. In certain places piles are sunk with a preliminary steaming of the frozen soil. This method is particularly effective if a steam leader is used in order that a bore hole of a given diameter may be sunk without thawing a large amount of soil.

The use of column foundations, that is, posts with shoes in the case of construction according to Principle I, is limited because of the difficulties connected with working in frozen soils, the difficulty of preserving the frozen state of soil when working during the summertime, and so forth. However, in recent years there has been a tendency toward the use of column foundations, for example, in areas with gravel soils where it is difficult to drill holes for sinking piles, but where, however, there is powerful earth-moving equipment which makes it possible to dig out and backfill excavations efficiently and rapidly.

CREEP DEFORMATIONS

Frozen soils, as is known, possess clearly pronounced rheological properties -- the capability of developing creep deformation and of reducing their strength in the case of the long-term action of loads (Fig. 2). The bearing capacity of a foundation (ϕ) in condition (4) determines the maximum long term resistance of the foundation, that is, that load, before exceeding which, the deformation of the soil damps out and failure does not take place. Upon exceeding this load, undamped creep, leading to failure over the course of time, arises. Correspondingly, a calculation according to condition (4) involved the determination of safe load (N), the action of which over the course of the service life of the structure does not lead to failure of the foundation or loss of stability.

When condition (4) is observed the deformations will be of a damped nature, and these deformations in the case of low temperature (solid frozen) soils will be relatively small, calculation according to condition (4) at the same time provides for fulfillment of condition (5). For high temperature, plastic frozen soils, even undamped deformations may reach a

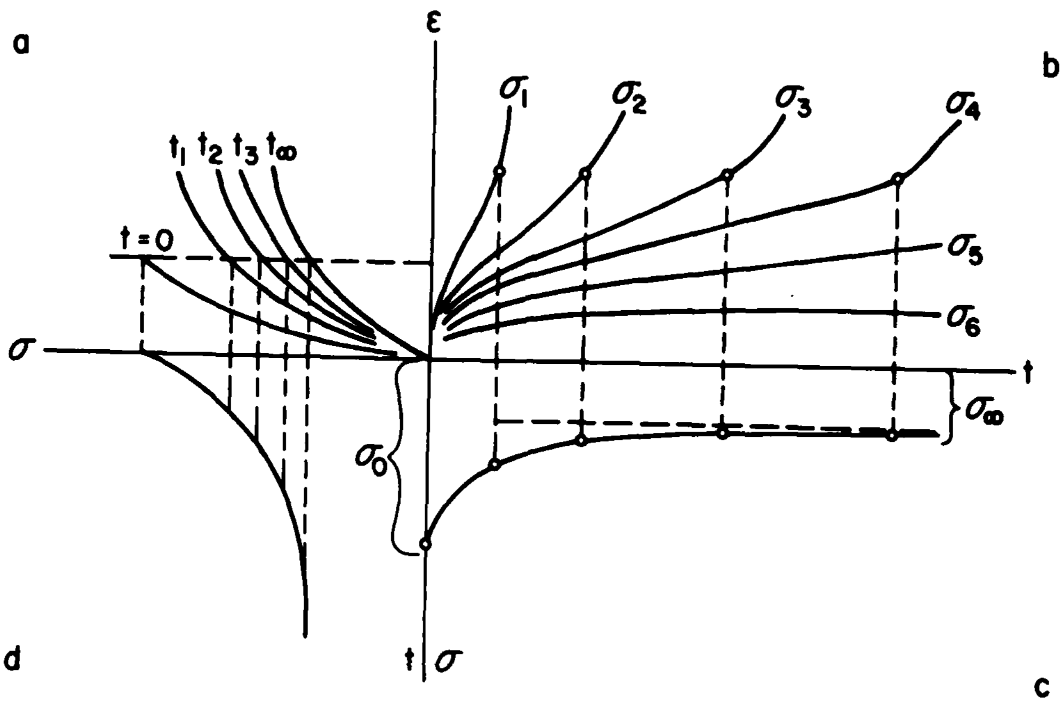


Figure 2. Diagram of the deformation of frozen soils.
a - relationship between stress and deformation at different moments of time (isochronous curves), b - development of deformations in time in the case of different stress (creep curves), c - reduction of strength depending on the time before failure (long term strength curve), d - weakening of stresses in time (relaxation curve).

significant magnitude, and therefore verification of the observation both of condition (4) and of condition (5) is necessary for the above-mentioned soils.

Indentation tests performed in the laboratory and under field conditions /10/ showed (Fig. 3) that the curves of the relationships between the settling (ζ) and load (P) are analogous to the isochronous curves in Fig. 2a. In the first place, the S-P relationship is clearly nonlinear and in the second place, the form of this relationship is determined by the duration of the loading stages. It is obvious that for a calculation it is

necessary to use the SP curve which corresponds to maintaining loading stages until the complete cessation of settling (curve P, Fig. 3c). On the basis of this curve it is possible to determine the limiting load (P_∞), the average value of which from several repeated experiments corresponds to the so-called normative soil pressure under the base of the foundation, $R^H = \frac{1}{n} \sum_{i=1}^{n} P_{\infty(i)}$.

The theoretical value R^H may be determined according to formula /3/:

$$R^H = 5.7\ C^H_{ek} + \gamma^H h, \tag{6}$$

where $C^H_{ek} = \frac{1}{n} \sum_{i=1}^{n} C_{ek(i)}$ is the normative value of the equivalent cohesion of the frozen soil, determined from "n" repeated experiments, γ^H is the normative value of the volume weight of the soil, h is the depth of the foundation. The value C_{ek} is the maximum long term value of the equivalent cohesion, that is, a generalized strength characteristic taking account of the cohesion itself and of the internal friction of the frozen ground. This value may be obtained from a sphere indentation test according to N.A. Tsytovich's method /3/, thus:

$$C_{ek} = 0.18\ \frac{P}{\pi d S_\infty} \tag{7}$$

where P is the load on the spherical stamp, d is the diameter, S_∞ is the stabilized penetration depth of the spherical stamp.

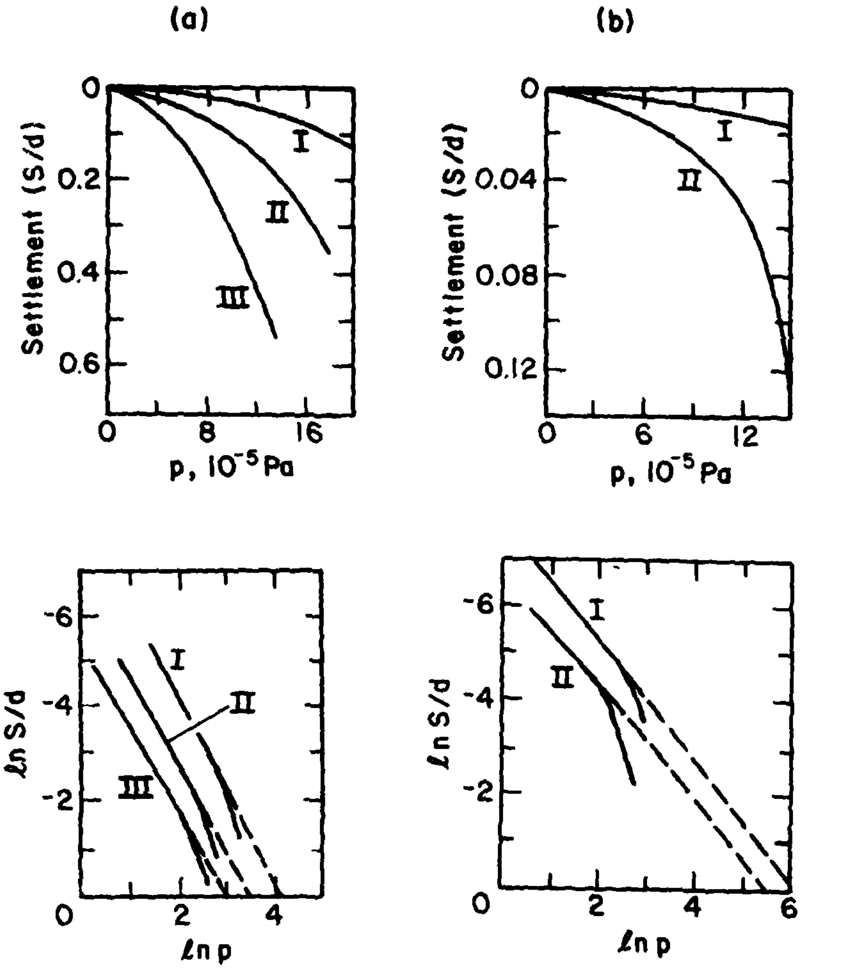
With a certain approximation it is possible to assume $C_{ek} = 0.5\ \sigma_{com}$, where σ_{com} is the long term strength limit for uniaxial compression, determined on the ordinary uniaxial compression instrument or according to the accelerated method, on a diamometer /3, 10/.

BEARING CAPACITY: COLUMNAR FOUNDATIONS

In evaluating the bearing capacity of a columnar foundation it is possible to take account of the normative shear resistance of the soil along the lateral edges of the foundation shoe resulting from the soil adfreezing to the edges, in addition to the soil pressure under the base of the foundation R_{cM}. Methods of determining R_{cM} will be discussed below. Correspondingly, the bearing capacity of a columnar foundation will be equal to (Fig. 4a):

$$\phi = m(RF + R_{cM}F_{cM}), \tag{8}$$

where R and R_{cM} are the calculated values of the pressure on frozen soil under the base of the foundation and the shear resistance of the frozen soil around the lateral edges of the foundation equal to $R = R^H/k_r$,



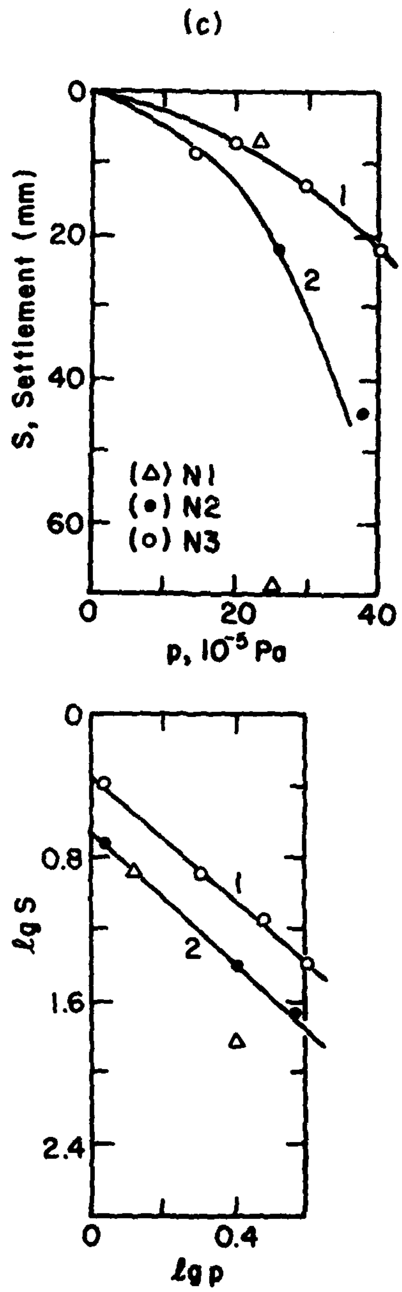


Fig. 3. Relationship between the settling S and the load P in ordinary and logarithmic coordinates in the case of different intervals of time Δt of maintaining loading stages. a - laboratory experiments (stamp d = 71 mm), sandy loam θ = -0.4°C, Δt = 5 min (I), 24 h (II), 504 h (III); b - large scale laboratory experiments (stamp d = 505 mm); compact clay, θ = - 0.6°C, Δt = 30 min (I), 72 h (II); c - field experiments (stamp d = 705 mm), sandy loam, θ = -0.1, -0.2°C, Δt = 48 h (I), before stabilization of settling (II).

$R_{cM} = R^H_{cM}/K_r$ ($K_r > 1$ is the safety factor); F and F_{cM} are the area of the base of the foundation and the area of the adfreeze of soil with the lateral edges of the shoe, m > 1 is the coefficient of the working conditions.

The values of R and R_M recommended for the calculations for different soils (depending on their temperature) are presented in the standards /1/. The values of the coefficients m and K_r also are indicated there.

BEARING CAPACITY: PILE FOUNDATIONS

Pile foundations, frozen or driven into leader holes, function as hanging piles, that is, they transmit a load to the ground as a result of the forces of resistance of the ground to shear along the lateral surface of the piling R_{cM}, and the pressure on the ground under the bottom end of the piling R (Fig. 4c). The value of R_{cM} corresponds to the maximum long term strength of the adfreezing of the soil with the piling, resulting from the cohesion forces C and the forces of friction of the soil along the piling, in turn depending on the coefficient of friction f and the forces of radial compression of the surrounding soil σ_r, therefore:

$$R^H_{cM} = C^H + F^H \sigma_r. \tag{9}$$

The radial stress σ_r arises as the result of the bulk expansion of the soil slurry in the bore hole upon freezing, the lateral pressure of the soil, and also as the result of the development of forces of soil compression when the piling is driven into a bore hole of small diameter. The ratio between the terms C^H and $F^H \sigma_r$ in formula (9) and the value of R^H_{cM} depends, thus, on the form and temperature of the soil and the soil solution, the material of the piling, and the method of sinking it.

The value of R^H_{cM} may be obtained either directly from field testing of the piles (without taking account of the work of its lower end), or from laboratory tests on a special instrument (A.V. Sadovsky), making it possible to determine the shear resistance of the soil frozen to the material under investigation, with simultaneous action of normal stress σ_r/3/.

The normative pressure under the lower end of the piling R is determined either directly from field testing of the pile with a separate determination of R and R_{cM} or from a solution of the limit equilibrium of a sunken stamp.

The bearing capacity of a single pile under the influence of a vertical load is determined by the formula

$$\phi = m \left(RF + \sum_{i=1}^{n} R_{cM(i)} F_{cM(i)}\right), \tag{10}$$

where $R = R^H/K_r$ and $R_{cM} = R^H_{cM}/K_r$ are the calculated values of R and R_{cM}, F

and F_{cM} are the cross-sectional area of the piling and the area of the freezing of soil with the surface of the piling, and m is the coefficient of the working conditions of the pile, taking account of the method of sinking it into the ground. The recommended values of R and R_{cM} for different soils, depending on their temperature, and also the values of m are presented in the standard /1/, being given in relation to the depth of the placement of the pile; the value of R increases with an increase in depth.

Formula (10) is identical to formula (8). The difference consists only in the fact that formula (10) takes account of the variation of R_{cM}, With depth of the soil, caused by the variation of its temperature, and also by the possible difference in the bedding of the soils. Taking account of these variations is important when calculating pile capacity, considering the great depth of their immersion in the permafrost layer.

SOIL TEMPERATURE DETERMINATION

Determination of the theoretical temperature θ, that is, the temperature which corresponds to the theoretical values of R and R_{cM} in formulas (8) and (10), is an important problem. As is known, the soil temperature above the depth of zero amplitudes (arbitrarily equal to 10 m) varies both with depth of the soil and in time in accordance with the variation of the temperature of the outside air. A method of taking account of this factor in determining the bearing capacity of foundations, including piling foundations, is discussed in S.S. Vyalov's report to the II International Conference on Permafrost (Yakutsk, 1973). However, the safety coefficient makes it possible to determine the bearing capacity of a foundation on the basis of accounting for the maximum average monthly temperature of the frozen ground at a given depth. In this case it is not the natural ground temperature which is considered, but that which will stabilize under the influence of an overlying building.

G.V. Porkhayev has developed a method of thermophysical calculation, introduced into the standards /1/, according to which the thermal conditions of the foundation of a structure are given beforehand, and may be distinguished from natural conditions. Thus, in the case of plastic frozen soils it is envisioned that their temperature will be lowered in the process of utilization as a result of the appropriate structural solution of a ventilated crawl space, and the operating conditions; and where necessary, a result of preliminary cooling of the soil, the foundation, the use of thermo-piles, and so forth. The expected distribution of the maximum average monthly soil temperature according to depth (Fig. 4) in relation to a depth of 10 m in natural conditions, and the average annual soil temperature on its upper surface (established during the utilization of the structure), may be determined on the basis of such a thermophysical calculation. The calculating formulas and auxiliary tables are presented in /1/.

In the case of uniform soils the design temperature may be averaged along the length of the piling, assuming it to be equal to a certain θ_z equivalent to the temperature $\theta_z = \theta_{ek}$ = const., so that the

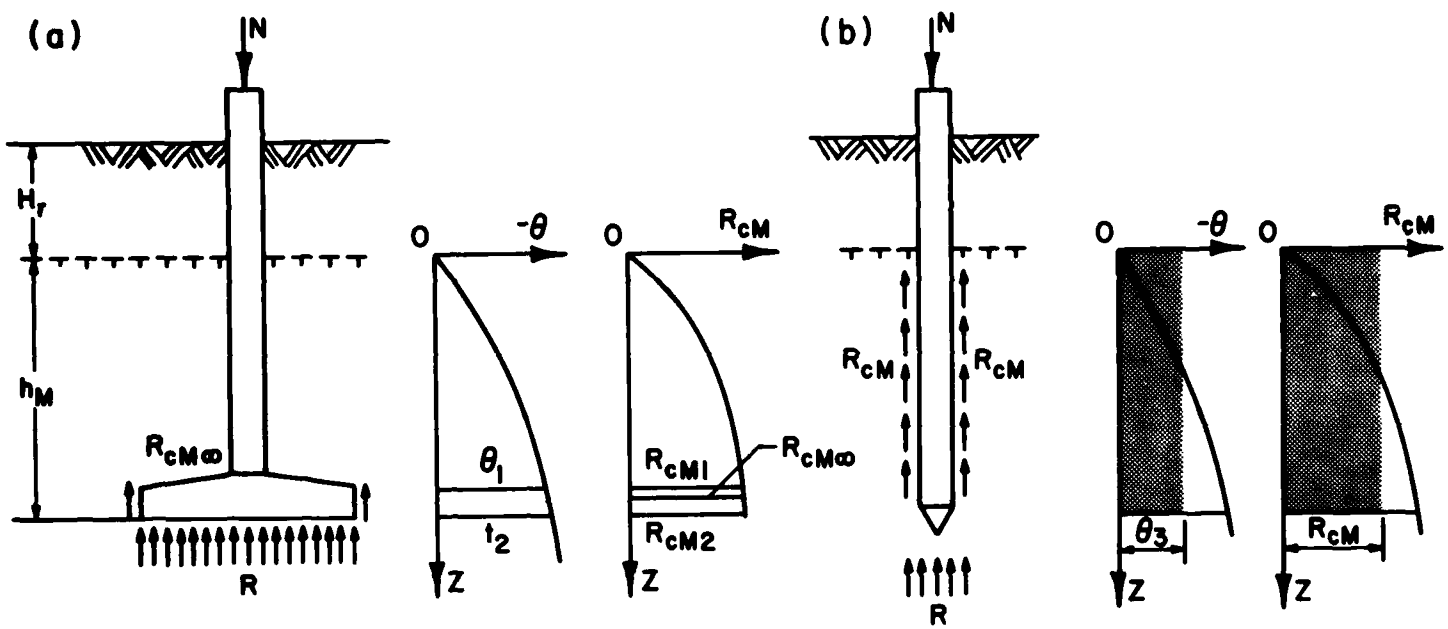


Figure 4. Calculation procedures for determining the bearing capacity of columnar (a) and piling (b) foundations.

area of the curve of R_{cM} for the value θ_{ek} = const. equals the area of the curve with variable values of θ_z (see Fig. 4). In this case the value of the bearing capacity of the piling is determined according to formula (8), substituting the values of R_{cM} corresponding to θ_{ek} into it.

We note that if the bearing capacity of the piles is found directly, according to the results of field tests, then it is necessary to take account of the possible deviation of the soil temperature during the testing period θ_{exp} from the planned value θ_{pl} when determining it. In this case the value of ϕ is defined as:

$$\phi = KP, \tag{11}$$

where $P = P^H/K_r$ is the calculated resistance of the pile to the test load, $P = P_\infty$ is the normative maximum long term load value determined from experiments, $K_r = 1.1$, $K = \phi_{exp}/\phi_{pl}$ is a coefficient considering the difference of the working conditions of experimental and planned pilings, ϕ_{pl} and ϕ_{exp} are the bearing capacities of planned and test piles determined according to formula (8) or (10) for values of R and R_{cM} corresponding to the temperatures θ_{pl} and θ_{exp} respectively, and taken according to tables in the standards /1/.

The shear resistance of frozen soil along the lateral surface of a foundation (adfreezing strength), is less than the shear resistance of a soil along the soil (which is due to the presence of an ice film on the soil-foundation surface contact). Therefore, the above mentioned contact is a weakness surface in piling-soil system, and shear takes place along it. However, for piles placed in holes filled with a soil slurry it is possible to use a sand-clay or, even better, sand-lime slurry such that the strength of its adfreezing to the pile will be greater than the shear resistance of the frozen solution along the surrounding ground. This particularly pertains to cases where the frozen soil is weak, for example, peaty, highly icy, saline, and so forth. In these cases a calculation of the bearing capacity of a piling may be performed from the equal strength condition:

$$\phi_1 = \phi_2, \tag{12}$$

where ϕ_1 is the bearing of a pile resulting from the shear resistance of a solution along the contact with the piling $R_{cM(1)}$ and ϕ_2 is the bearing capacity of the pile due to the shear resistance of the solution in contact with the surrounding ground $R_{cM(2)}$ (Fig. 5). The values of ϕ_1 and ϕ_2 for uniform soils are determined by a formula of the form (8):

$$\phi_{(1,2)} = m\,[RF_{(1,2)} + R_{cM(1,2)}\,F_{cM(1,2)}], \tag{13}$$

where $F_{(1)}$ and $F_{(2)}$ are the cross sectional areas of the pile and the cross sectional area of the bore hole, $F_{cM(1)}$ and $F_{cM(2)}$ are the area of

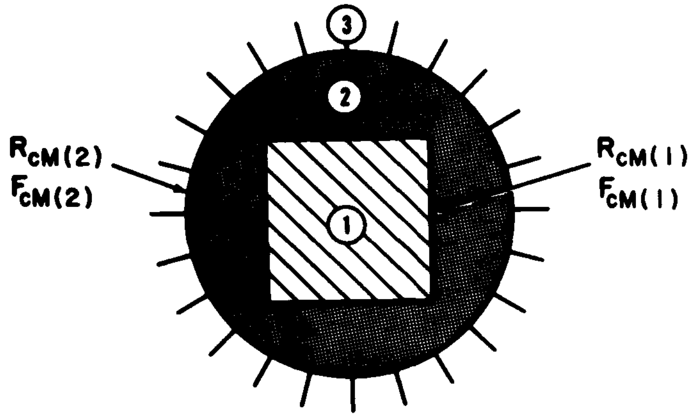


Figure 5. Set-up for equal strength calculation: 1 - piling, 2 - frozen soil solution, surrounding frozen soil.

the lateral surface of the piling and the area of the lateral surface of the bore hole.

Although $R_{cM(1)} = R_{cM(2)}$, $F_1 < F_2$ and $F_{cM(1)} < F_{cM(2)}$. Therefore, it is possible to choose a ratio of the above mentioned values such that condition (3) will be satisfied and, therefore, the bearing capacity of the piling foundation will be raised.

A calculation of foundations in plastic frozen soils with respect to deformations in the general case must take account of the nonlinear connection between the load, the settling (see Fig. 3) and the development of settling in time (Fig. 6).

The curves shown in Figs. 3 and 6 are obtained from long term experiments conducted in Igarka /10/.

The field experiments presented in Figs. 3c and 6 involved the indentation of three stamps (d = 705 mm) in plastic soils and lasted for 19 years.

As is obvious, the settling of stamps in the case of loads of from 2.5 to 4.0 x 10^5 Pa developed continuously until cooling of the foundation soils was performed.

The law governing the development of settling in the case of a variable load and taking account of the variation in time of the temperature of foundation soils may be described by the following formula /10/

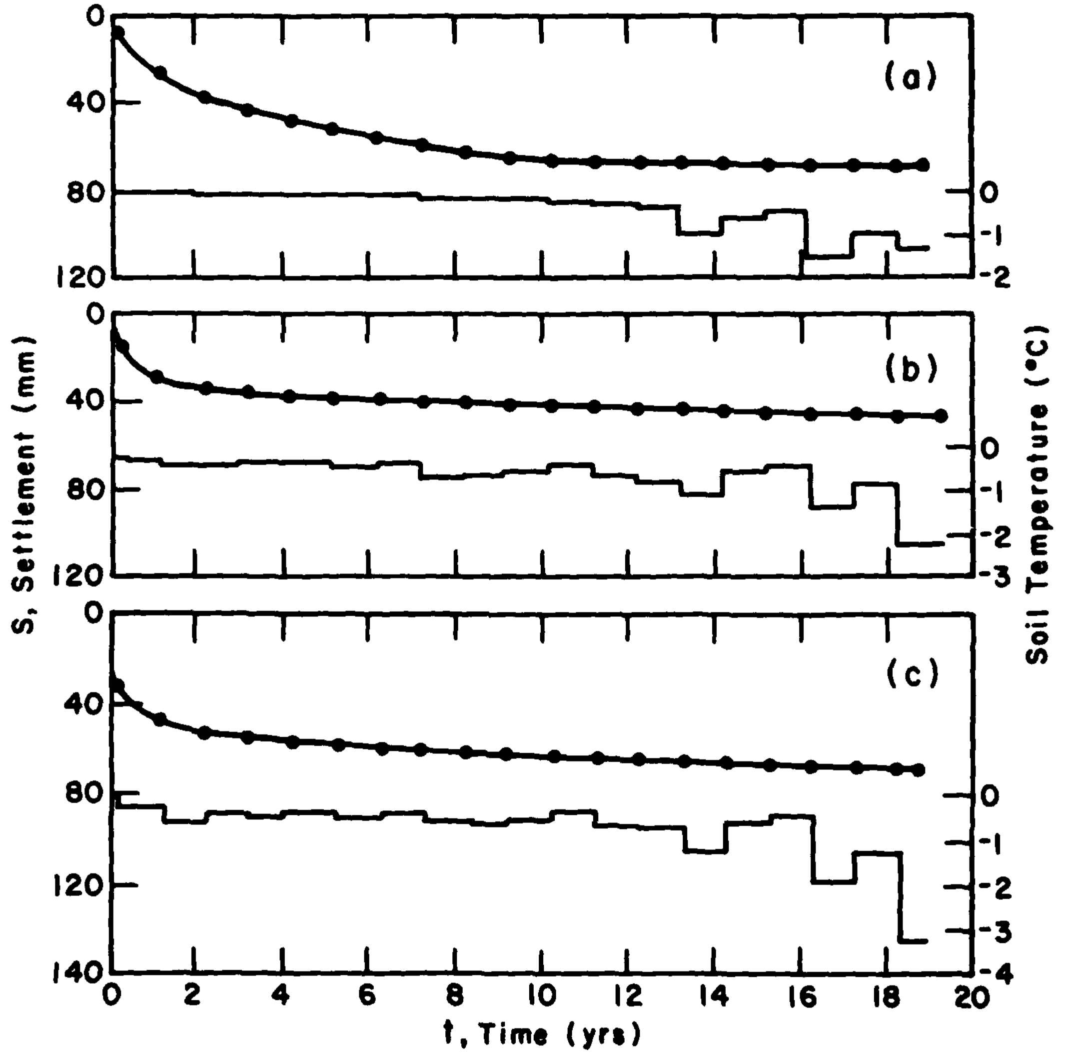


Figure 6. Development of settlings of test stamps (d = 705 mm) in plastic frozen soils under constant loads: a) - p = 2.58; b) - p = 3.75; c) - p = 4.0×10^{10} Pa.

$$S/d = (1-\mu^2)w \frac{P(t)}{A(\theta,t)} + \beta\nu \int_0^t \frac{P(\xi)}{A[\theta(\xi)]}^{1/m} (t-\xi)^{\beta-1}\, d\xi \ , \tag{14}$$

where: d is the diameter or reduced width of the base of the foundation
w is the coefficient depending on the form and ridigity of the foundation ($w = 0.79$ for a round rigid foundation)
$P(t)$ is a load time variable
$A(\theta,t)$ is the coefficient of deformation, depending on the temperature θ, which in turn may be variable in time such that $A(\theta,t) = A_o + b/\theta\ (t)/\theta$
m, β, ν are soil parameters
t is time
ξ is a integration constant.

In order of approximation, the average annual temperature value at a given depth, $\sigma_{av(z)}$ = const, may be taken as the calculating temperature. If, in addition, the load is constant, then formula (14) acquires the following simple form:

$$S/d = BP^{1/m}\ (1 + \nu t^{\beta}), \tag{15}$$

where $B = W(1-\mu^2)A^{1/m}$.

For an approximate estimate of the ultimate stabilized settling it is possible to start from the linear connection (2) between the settling and the load and to use the well-known formula for the linear summation of settlings from pressures under the center of the foundation, therefore:

$$S = \sum_{i=1}^{n} \delta_{M(i)}\ P_i h_i, \tag{16}$$

where: n is the number of layers into which the soil stratum is divided
h_i is the thickness of this layer
$P_i = \alpha P_0$ is the pressure at the center of the i-th layer
P_0 is the pressure transmitted to the soil under the base of the foundation
α is the coefficient characterizing the pressure variation according to the depth of the soils
$\delta_{M(i)}$ is the relative compressibility of the i-th layer of soil according to formula (2).

The settling of a pile foundation in frozen soils is determined on the basis of the results of field experiments according to the formula

$$S/U = \left(\frac{N}{A_{cb} U h_M}\right)^{1/m} (1 + \nu t^{\beta}), \tag{17}$$

where: N is the calculated load on a pile satisfying the condition (4)

U and h_M are the perimeter of the cross-secion of the piling and the depth of its placement in permafrost

A_{cb}, m, and ν are parameters of the deformation of the permafrost, whereby $A_{cb} = f(\theta)$

t is the design time equal to the service life of the structure.

Formula (17) is analogous to formula (15), the only difference being that the parameter A_{cb} is an empirical value reflecting the deformation properties of the piling-frozen soil system, and obtained directly from field testing.

DESIGN OF THAWING FOUNDATIONS

Evaluation of Deformation Properties of Thawing Ground

We shall examine a calculation of thawing foundations, based on condition (5).

The compactability of soils during thawing is their most important property, determining the content of engineering surveys at the site, the principle of the utilization of soils as a foundation, the necessity for preconstruction preparation of the area, and the reinforcement of buildings or structures. At different stages of surveying and planning these properties of frozen soils may be evaluated with different degrees of accuracy. An initial evaluation of soils is performed for the purpose of determining the content of experimental engineering-geological studies in an area, in particular, for determining the necessity of laborious investigations, of soil compressibility in the case of thawing. It is possible to determine if soils are subject to settling or, poorly compressible (according to their simplest physical characteristics), the moisture or unit weight of the skeleton, comparing their values with standard values established for a given region. Such data have been published several times, for example, in the Handbook for Construction on Permafrost /2/.

A more accurate evaluation of soils is achieved by means of determining the maximum possible value of relative compression, according to approximate formulas, the most common of which are the following /1/ for sandy soils:

$$\delta_t = (\gamma_t - \gamma_m)/\gamma_t, \qquad (18)$$

and for clay soils (with a pressure of more than 1 kg/cm^2)

$$\delta_t = 1 - \gamma_m \left[\frac{1}{\gamma_s} - \frac{1}{\gamma_w} (W_p + K_g J_p)\right] , \qquad (19)$$

where γ_m and γ_t are the unit weights of the soil skeleton in the frozen and thawed compacted states

γ_s and γ_w are the specific weights of the soil and water particles

W_p and J_p are the moisture on the liquid and plastic limits

K_g is a coefficient depending on the pressure and J_p, the values of which are given in tables /1/.

The relative compression determined according to these formulas usually is used for choosing the principle of utilization of soils as foundations, for which the expected settling is calculated approximately as:

$$S = \sum_{i=1}^{n} [\delta_{t(i)} (1 - L_{ci}) + K_{bi} L_{ci}], \tag{20}$$

where: n is the number of layers into which the stratum of thawing soils is divided, differing by the value of the relative compression σ_i and the ice content due to the inclusion of ice L_{ci}.

L_{ci} is the difference between the total ice content of the i-th layer of soil and a sample taken from this layer for an experimental determination of the soil characteristics

K_{bi} is a coefficient taking account of the incomplete closure of macro-pores in the soil during its thawing, the values of which are presented in the standards /1/.

A further improvement in the accuracy of the method of evaluating soils is achieved by a more perfect determination of the relative compression with substitution of this value into (3).

The coefficients A and a found in formula (3) are determined for each i-th layer of soil. Previously these coefficients were determined primarily according to the results of laboratory tests in soil compression testers /3/. However, the practice of recent years has shown that such determinations, as a rule, yield exaggerated values of the compressibility characteristics, especially for incoherent soils, for which they may prove to be exaggerated by more than an order of magnitude, and therefore the determination of A and a in field conditions with a hot stamp /3/ has become more and more common. In this case formula (18) retains its value, but the difference in ice levels L_{ci} proves to be equal to zero. During recent years the stamp testing technique has undergone certain changes. Peripheral heaters have begun to be used for equalizing the boundaries of thawing under the stamp and excluding the possibility of soil adherence along the perimeter of the thawing zone. This led to the necessity of changing the method of analyzing the results of the test for determining the value of the compressibility factor. The relationship between the deformation modulus, E_t, and the load on the stamp makes it possible to calculate the compressibility factor a_t $(1-2M^2)/(1-\mu)$ E_t, where μ is the Poisson coefficient /4/ found from a solution of the problem using the methods of the theory of elasticity on the indentation of a round stamp into a soil cylinder of large diameter, contained in a rigid holder, and resting on the same foundation.

SOIL THAW DEFORMATION CHARACTERISTICS

In the final stage of planning we have the problem of satisfying condition (5) as precisely as possible, where S, the expected, and S_{lim}, the limiting, deformation entering into this condition are understood as matrices including the absolute motions and relative deformations of the foundations, estimated by the irregularity of settling of two adjacent foundations, the pitch, and the relative downwarp or upwarp and so forth /5/.

The expected deformations from loads, determined without taking account of the ridigity of the structures of the buildings, including the foundations, are calculated initially. In addition, the pressure on the foundation is limited by the value of the theoretical resistance, determined, as usual, for unfrozen soils /5/, by taking account of those characteristics that soils acquire after thawing. The calculated deformations are compared with the limiting ones, the values of which are presented in the design Standards for the most common structural types of buildings and structures /5/. If the inequality is not observed and is is not possible to bring S and S_{lim} into agreement by means of the appropriate preparation of the base, regulation of its thawing, increasing the depth of the foundations and other planning measures, then it becomes necessary to reinforce the components of the buildings or structures, considering their joint deformation with the foundation. The settling of a thawing foundation is determined by the layer-by-layer summation of the compression of the individual layers, the calculating schemes being used in the form of a linearly deformable half-space or a layer of finite thickness. The choice of scheme depends on the thickness of the layer of frozen soils and the expected depth of thawing. If the thawing spreads throughout the frozen layer, as takes place when the frozen ground is in blocks or is at a fairly great depth, then the first scheme is used; in other cases, the second is used.

However, in practice the choice of a calculating procedure has meaning only in the case where the thawing spreads from the base of the foundation to a depth not exceeding three times its width, since in the case of a greater thickness of the layer being compressed the influence of an underlying rigid surface on the settling becomes insignificant. With any of the above two procedures the settling is defined as the sum of two components:

$$S = S_o + S_p \tag{21}$$

where S_o is the component of the settling, independent of the load transmitted to the foundation by the building or structure, resulting from the compaction of the soils upon their thawing under the influence of their own weight and S_p is the component caused by the compaction of the thawed soils under the influence of constant and temporary loads transmitted by the base of the foundation to the soil stratum.

These components may be determined according to formulae /1,6/ corresponding to the first and second calculation procedures, hence:

$$S_o = \sum_{i=1}^{n} [(A_i + a_i p_{\delta i})(1 - L_{ci}) + K_{\beta i} L_{ci}] h_i, \qquad (22)$$

and,

$$S_p = P_o bM \sum_{i=1}^{n} a_i (K_{\mu,i} K_i - K_{i-1} K_i). \qquad (23)$$

In these formulas, in addition to formula (2), the following symbols are used (Fig. 7):

$P_{\delta i}$ and P_o are the average pressure in the i-th layer, from the weight of the soils themselves, and the additional pressure transmitted to the base of the foundation, with width b

M is a coefficient reflecting the influence on the base of a rigid layer, the values of which are taken according to tables compiled where $\mu = 0.30$ (in the case of calculation using the first procedure M = 1)

$K_{\mu,i}$ and $K_{\mu,i-1}$ are correcting factors, differing from unity, when $\mu \neq 0.30$

K_i and K_{i-1} are coefficients reflecting the stressed state in the i-th layer, determined for each of the two calculating procedures (according to different tables) depending on the shape of the base of the foundation and the relative distance from it to the roof and base of the layer.

At the present time there is a sufficient set of tables which makes it possible to determine the settling of a rigid foundation and also the settling of the central, corner, and edge points of the base of a flexible foundation, and to account for the "off center" application of a load and the different depth of thawing under the edges of the foundation, necessary for the calculation of its pitch (i). Therefore:

$$i = \frac{S_a - S_b}{b} \quad , \qquad (24)$$

where S_a and S_b are the settling of the edge points of the base and b is its dimension parallel to the plane of the pitch.

In calculating settlings according to formulas (22) and (23) the depth of the compressible zone of the base is taken as equal to the theoretical depth of thawing with the exception of cases where the thawing spreads throughout the frozen soils; then the compressible zone is determined as is done for unfrozen soils.

BASE DESIGN: JOINT-BUILDING COMPONENT INTERACTION

In the components of a building or structure which provide for the rigidity in the longitudinal and transverse directions, there arise supplementary forces as a consequence of the spatial irregularity of the thawing cup of soils, as well as the nonuniformity of their composition. A building is considered as resting on a base distorted as a result of the

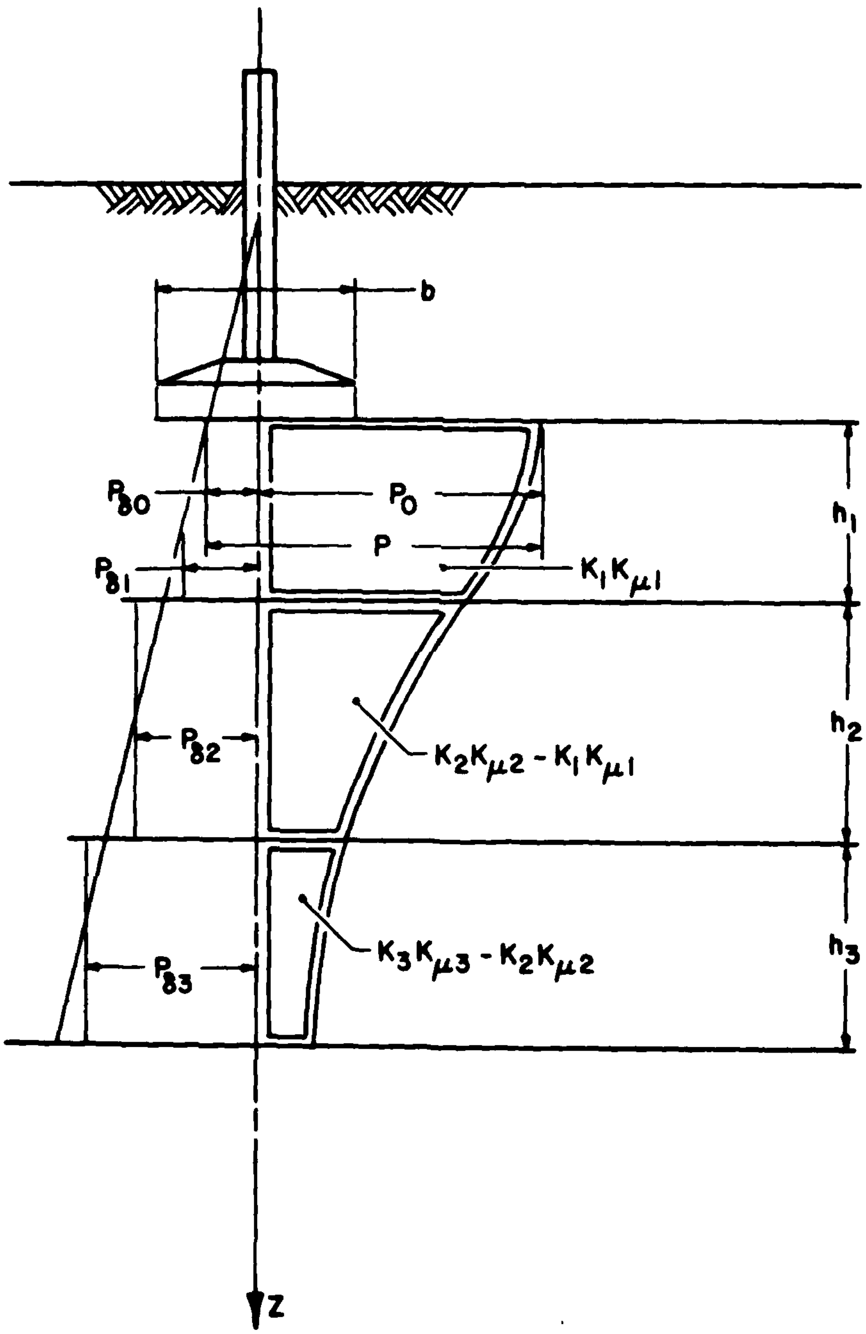


Figure 7. Diagram for calculating settlings.

compaction of the thawing soils under the influence of their own weight. The settling of the base at any point is known and equal to "S_o." Under the influence of constant and temporary loads there are additional deformations which are found by means of solving the statically indeterminate problem of the compatibility of the deformations in the building - base system /2/. In solving this probelm we consider the possibility of the appearance of three different zones of the base:

1. a zone of zero reactions where $q = 0$, and the movement of the foundation w, variable along its length, is less than the settling of the base S_o, that is, $w < S_o$

2. a compaction zone where the reaction is different from zero, but does not exceed the limiting load q_{lim}, that is, $0 < q < q_{lim}$, and the movement of the foundation follows the settling of the base; here, $q = c(w - S_o)$ and $S_o < w \leq S_o + q_{lim}/c$ where c is the rigidity characteristic of the base, which may be variable along the length of the foundation.

3. a zone of plastic deformations where the reactions of the base ($q = q_{lim}$) and the movements of the foundation are limited only by the rigidity of the components.

The value of C at each point along the length of a band foundation is found as a value inverse to the compaction settling component from a single load, that is

$$C = g/S_{(g)},$$

where g is the linear load on the base, and $S_{(g)}$ is the settling determined by formula (23).

The reactive pressures on the base of a band foundation in the general case are determined for three-dimensional or two-dimensional models of a building, using a special form computer calculation /2/. In cases where the rigidity of the underground components is negligibly small or very great, simple solutions, easily solved in planning practice (even without a computer), are obtained. Thus, for example, for a frame building it may be considered that the supplementary forces from nonuniform settlings are completely received by the band foundations, and this makes it possible to obtain a solution in closed form for the forces, relative downwarp, and pitch, in the case of a given bend and shear rigidity of the construction of the foundation /7/. The solution of the problem for a band foundation of the wall of an unframed building, the height of which is not less than its length, may be obtained under the assumption of absolute rigidity of construction. Initially, solutions are found under the assumption that in the middle of the wall $q_{av} > 0$, and along its edges $q_{cr} \leq q_{lim}$. If this condition is not satisfied, then the calculation is performed by the successive approximation method according to one of the procedures indicated in Fig. 8 until the results of determining the lengths of the sections with zero reactions, compaction, and limiting equilibrium differ insignificantly according to the two last approximations (by not more than a given value, usually taken as equal to 5%).

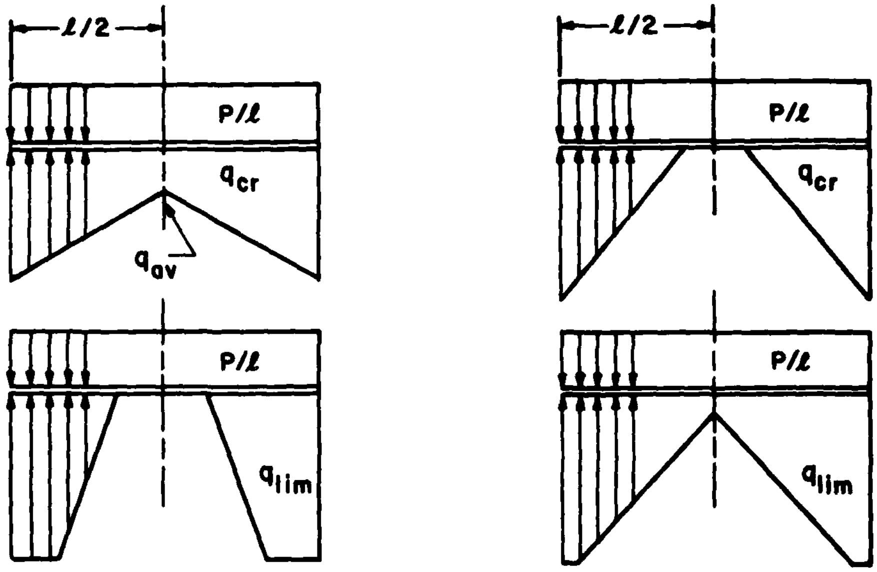


Figure 8. Diagrams of reactive pressures on the base of a foundation

CONCLUSIONS

Experience in using the method of calculating bases discussed here in planning practice, taking account of the rigidity of the components of buildings, has yielded sufficiently satisfactory results, although this problem is far from exhausted. Perfection of the technique of determining the deformative characteristics of soils, increasing the accuracy of the theoretical model of the base and taking account of the nonlinearity of the relationship between deformations and load (and also the values of the limiting load) are the principal tasks of the scientific research work in future years /8/.

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DESIGN OF FOUNDATIONS
IN AREAS OF SIGNIFICANT FROST PENETRATION

by - K.A. Linell[1], E.F. Lobacz[2] and H.W. Stevens[3]

INTRODUCTION

1. TYPES OF AREAS. For purposes of this paper, areas of significant frost penetration may be defined as those in which freezing temperatures occur in the ground to sufficient depth to be a significant factor in foundation design. Detailed requirements of engineering design in such areas are given in reference 34 and in the Arctic and Subarctic Construction series, TM 5-852-1 through 9(5-13).

Areas of significant frost penetration may be subdivided as follows:

a. Seasonal frost areas are those where significant ground freezing occurs during the winter season, but without development of permafrost. In North America significant seasonal frost occurs about 1 year in 10 in northern Texas. A little farther north it is experienced every year. Depth of seasonal freezing continues to increase northward with decreasing mean annual and winter air temperatures until permafrost is encountered. With still further decrease of air temperatures, the depth of annual freezing and thawing becomes progressively thinner.

The layer extending through both seasonal frost and permafrost areas in which annual freeze-thaw cycles occur is called the annual frost zone. In permafrost areas it is commonly called the active layer, but in this paper the term annual frost zone is used for sake of a single designation applicable for all frost regions. It is usually not more than 10 ft (3 m) thick, but it may exceed 20 ft (6 m). Under conditions of natural cover in very cold permafrost areas, it may be as little as 1 ft (0.3 m) thick. Its thickness may vary over a wide range even within a small area. Seasonal changes in soil properties in this layer are caused principally by the freezing and thawing of water contained in the soil. The water may be permanently in the annual frost zone or may be drawn into it during the freezing process and released during thawing. Seasonal changes are also produced by shrinkage and expansion caused by temperature changes.

b. Permafrost areas are those in which perennially frozen ground is found. When the ground temperature curve with depth at its warmest extreme is below freezing over a portion of its length, as in Figure 1, a permafrost condition exists. In North America permafrost is found principally north of latitudes 55 degrees to 65 degrees, although patches of permafrost are found much farther south on mountains where the temperature conditions are sufficiently low, including some mountains in the

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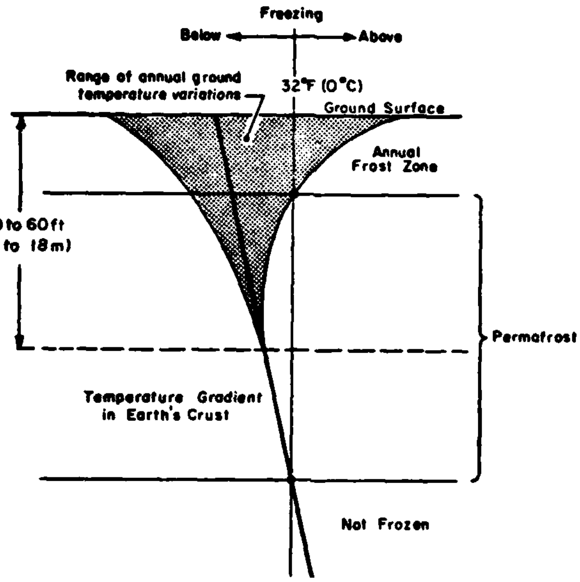


Figure 1. Typical temperature gradients in the ground.

contiguous 48 states. In zones of continuous permafrost, frozen ground is absent only at a few widely scattered locations, as at the bottoms of rivers and lakes. In zones of discontinuous permafrost, permafrost is found intermittently in various degrees. There may be *discontinuities* in both horizontal and vertical extent. Sporadic permafrost is permafrost occurring in the form of scattered permafrost islands. In the coldest parts of the Arctic the ground may be frozen as deep as 2000 ft (600 m).

The geographical boundaries between zones of continuous permafrost, discontinuous permafrost, and seasonal frost without permafrost are poorly defined, but are represented approximately in Figure 2.

A number of soil and other terms relating to frozen ground areas are defined or illustrated in Figure 3. Additional definitions of specialized terms are given in TM 5-852-1(5) and reference 23.

2. GENERAL NATURE OF DESIGN PROBLEMS. Generally, the design of foundations in a seasonal frost area follows the same procedure as where frost is insignificant or absent. However, the foundations are taken to depths below the reach of annual frost penetration, and precautions are taken where necessary to prevent uplift or thrust damage as a result of frost forces acting on the members which transmit structure loadings through the annual frost zone to the bearing or anchoring elements below. Thaw and settlement of frost-heaved material in the annual frost zone may occur differentially, and a very wet, poorly drained ground condition with temporary but substantial loss of shear strength is typical during thaw.

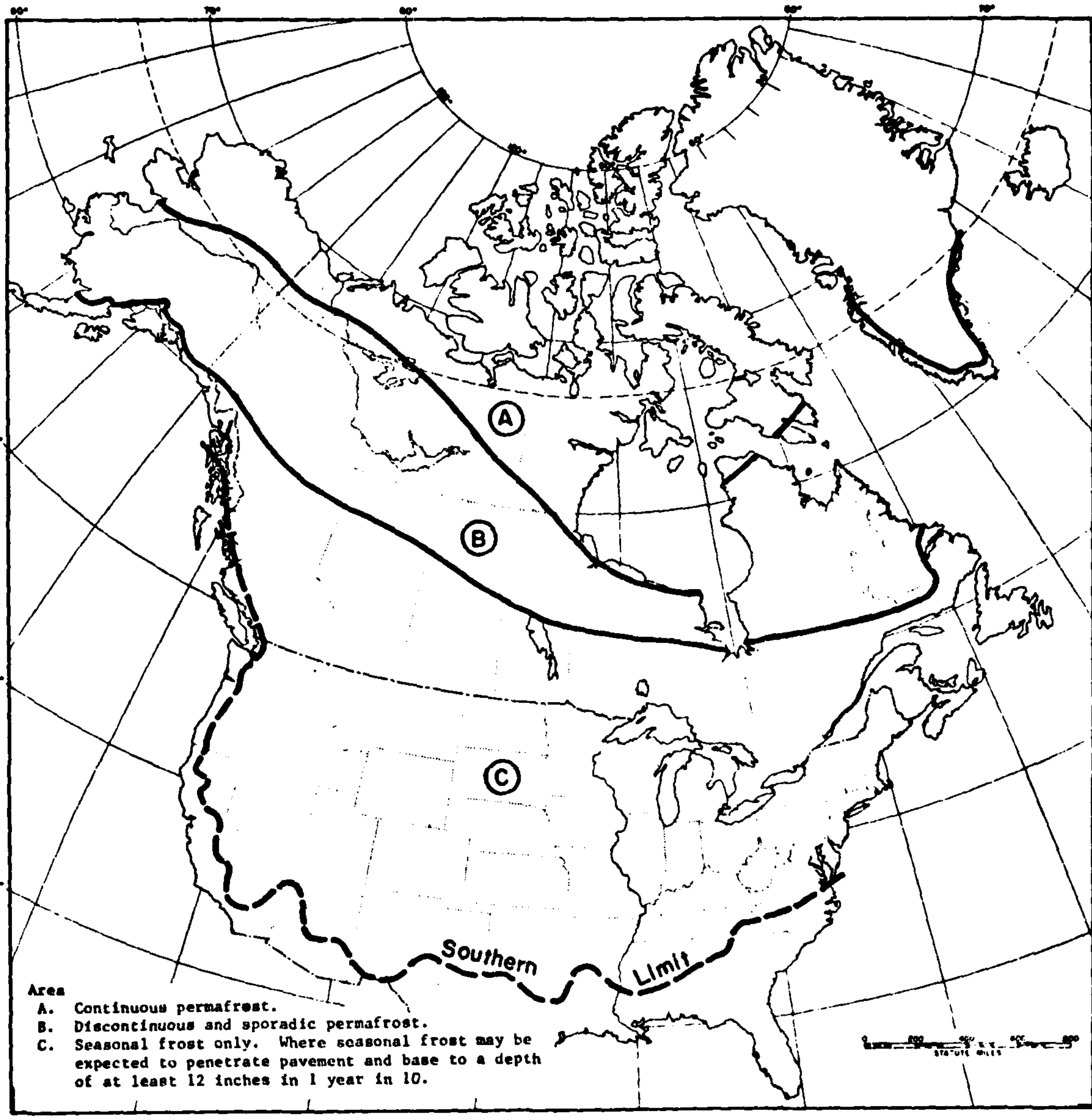
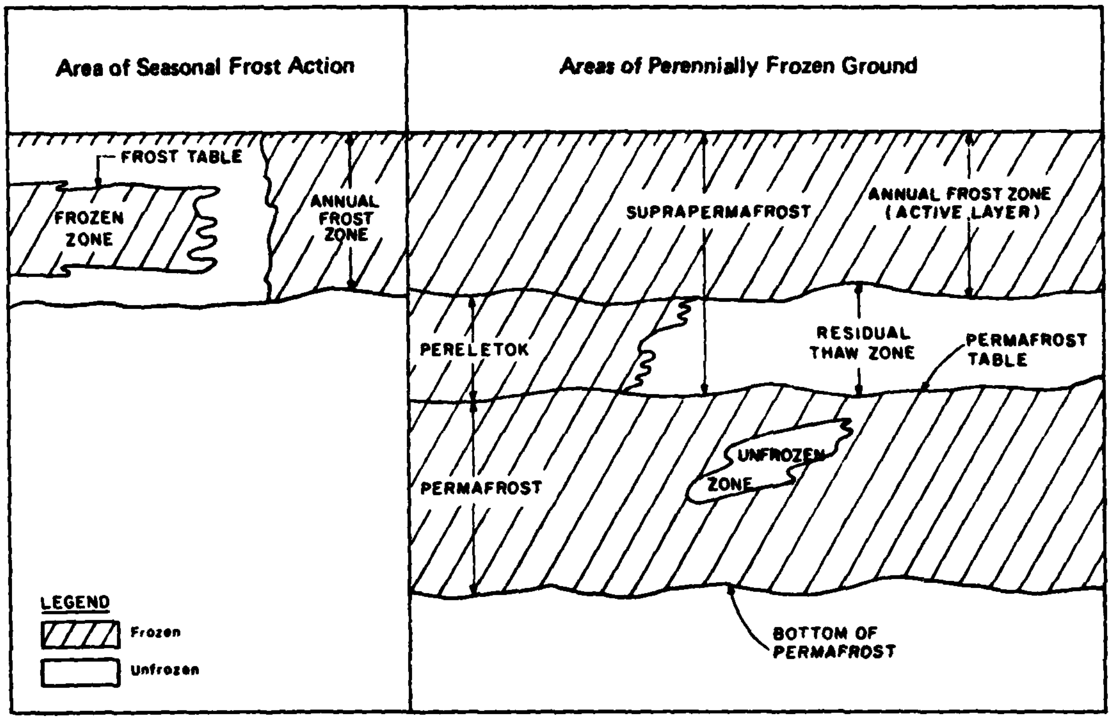


Figure 2. Frost and permafrost in North America.

In permafrost areas, the same annual frost zone phenomena occur. However, structures in these areas are either supported on top of the annual frost zone or are supported in the underlying permafrost zone, using piles or other means to transmit structure loads through the annual frost zone. In permafrost areas, heat flow from the building is a fundamental consideration, complicating the design of all but the simplest buildings. Any change from natural conditions which results in a warming of the ground beneath a structure can result in progressive lowering of



DEFINITIONS OF SOIL AND OTHER TERMS RELATING TO FROZEN GROUND AREAS*

Annual frost zone (active layer). The top layer of ground subject to annual freezing and thawing. In arctic and subarctic regions where annual freezing penetrates to the permafrost table, suprapermafrost and the annual frost zone are identical.

Excess ice. Ice in excess of the fraction which would be retained as water in the soil voids upon thawing.

Frost table. The surface, usually irregular, which represents the level to which thawing of seasonal frozen ground has penetrated.

Frozen zone. A range of depth within which the soil is frozen. The frozen zone may be bounded both top and bottom by unfrozen soil, or at the top by the ground surface.

Ground ice. A body of more or less clear ice in the ground.

Ice wedge. A wedge-shaped ice mass in permafrost, usually associated with fissure polygons.

Icing. A surface ice mass formed by freezing of successive sheets of water.

Permafrost. Perennially frozen ground.

Permafrost table. The surface which represents the upper limit of permafrost.

Pereletok. A frozen layer at the base of the annual frost zone which remains unthawed for one or two summers.

Residual thaw zone. A layer of unfrozen ground between the permafrost and the annual frost zone. This layer does not exist where annual frost extends to permafrost.

Suprapermafrost. The entire layer of ground above the permafrost table.

Figure 3. Definitions and illustrations of frozen ground terms.

the permafrost table over a period of years. This is known as degradation. If the permafrost contains ice in excess of the natural void or fissure space of the material when unfrozen, progressive downward thaw may result in extreme settlements of overlying soil and structures. This is very serious because subsidence from this means is almost invariably differential and hence very damaging to a structure. Degradation may occur not only from building heat but also from solar heating from surface water and groundwater flow, and from underground utility lines. Proper insulation will prevent degradation in some situations but where a continuous source of heat is available thaw will, in most cases, eventually occur.

Engineering problems may also arise from such factors as the difficulty of excavating and handling ground when it is frozen; soft and wet ground conditions during thaw periods; surface and subsurface drainage problems; thermal stresses and cracking of the ground in very cold areas; special behavior and handling requirements for natural and manufactured materials at low temperatures and under freeze-thaw action; possible ice uplift and thrust action on foundations; condensation on cold floors; adverse conditions of weather, cost and sometimes accessibility; and, in the more remote locations, limited local availability of materials, support facilities and labor.

Progressive freezing and frost heave of foundations may also develop under refrigerated warehouses and other facilities where sustained interior below-freezing temperatures are maintained. The design procedures and technical guidance outlined in this paper and in the cited references (4,36) may be adapted to the solution of these design problems.

FACTORS AFFECTING DESIGN OF FOUNDATIONS

1. PHYSIOGRAPHY AND GEOLOGY. Physiographic and geologic details in the area of the proposed construction are a major factor determining the degree of difficulty which may be encountered in achieving a stable foundation. For example, pervious layers in fine-grained alluvial deposits in combination with copious groundwater supplies from adjacent higher terrain may produce very high frost heave potential, but clean, free-draining sand and gravel terrace formations of great depth, free of excess ice, can provide virtually trouble-free foundation conditions.

2. TEMPERATURE. The most important factors contributing to the existence of adverse foundation conditions in seasonal frost and permafrost regions are cold air temperatures and the continual changes of temperature between summer and winter. Mean annual air temperatures usually have to be 2 to 8°F (1.1 to 4.4°C) below freezing for permafrost to be present, with the extremes about 0 to 12°F (0 to 6.7°C). Ground temperatures, depths of freeze and thaw, and thickness of permafrost are the products of many variables including weather, radiation, surface conditions, exposure snow and vegetative cover and insulating or other special courses. The properties of earth materials which determine the depths to which freezing and thawing temperatures will penetrate below

the ground surface under given temperature differentials over a given time are the thermal conductivity, the volumetric specific heat capacity and the volumetric latent heat of fusion. These factors in turn vary with type of material, density and moisture content. Figure 4 shows how ground temperatures vary in an area of substantial seasonal freezing (Limestone, Maine), and Figure 5 shows similar data for a permafrost area having a mean annual temperature of 26°F (-3.3°C) (Fairbanks, Alaska).

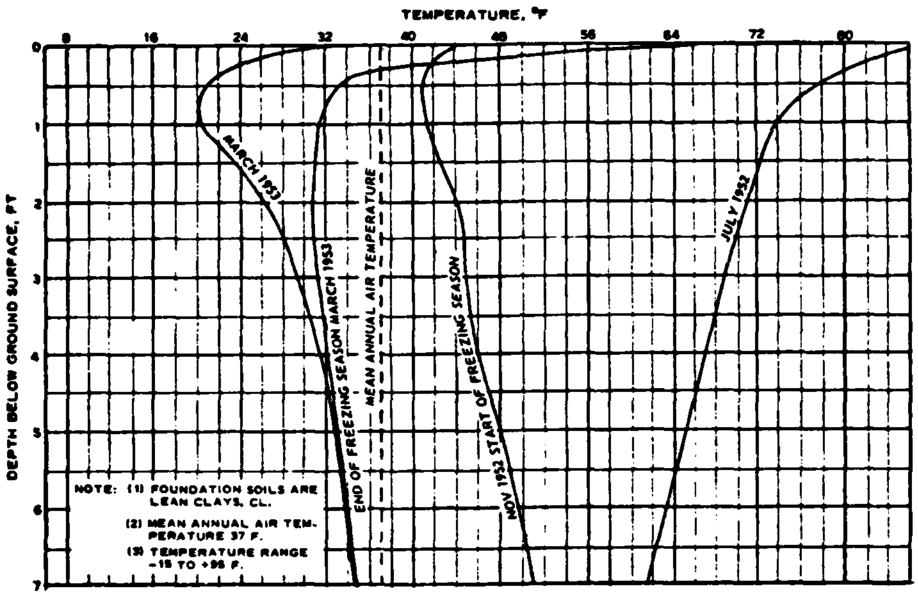


Figure 4. Ground temperatures during freezing season, Limestone, Maine.

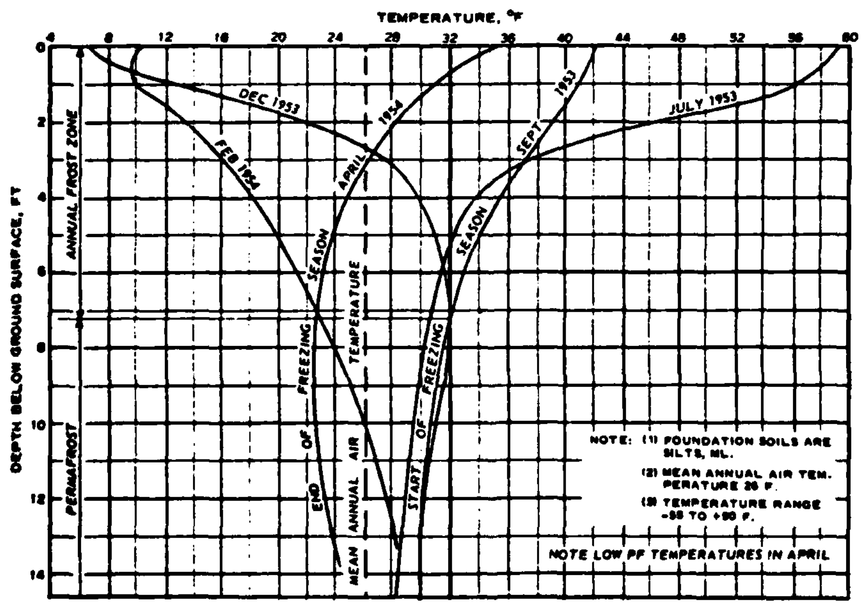


Figure 5. Ground temperatures during freezing season, Fairbanks, Alaska.

The more intense the winter cooling of the frozen layer in the annual frost zone and the more rapid the rate of frost heave, the greater the intensity of uplift forces on piles and foundation walls. The lower the temperature of permafrost, the higher the bearing capacity and adfreeze strength which can be developed, the lower the creep deformation rate under footings and in tunnels and shafts, and the faster the freeze-back of slurried piles. Dynamic response characteristics of foundations are also a function of temperature. Both natural and manufactured construction materials experience significant linear and volumetric changes and may fracture with changes in temperature. Shrinkage cracking of flexible pavements is experienced in all cold regions. In arctic areas, patterned ground is widespread, with vertical ice wedges formed in polygon boundaries. When underground pipes, power cables or foundation elements cross shrinkage cracks, rupture may occur during winter contraction. During summer and fall, expansion of the warming ground may cause substantial horizontal forces if the cracks have become filled with soil or ice.

For computation of seasonal freeze or thaw penetration, freezing and thawing indexes are used based upon degree-days relative to 32°F (0°C). If t°F is the average temperature recorded on a certain day, the number of Fahrenheit degree-days for that day is (t - 32); if the result is positive, there are degree-days of thaw; if the result is negative, there are degree-days of freezing. The cumulative algebraic sum of the individual degree-days of air temperature taken through the complete freezing season (when the average daily temperature remains generally below freezing) is the air freezing index. Similarly, a summation over the complete thawing season (when the average daily temperature remains generally above freezing) gives the air thawing index. The average daily temperature is usually found by averaging the maximum and minimum temperatures recorded for the day. When this does not give sufficiently representative values, the average of several temperature readings at equal time intervals during the day, usually hourly, may be used. Indexes may also be calculated from average monthly temperatures as described in TM 5-852-6(10). The word "average" is used for one particular interval of time. "Mean" implies the average of averages taken over several similar time intervals. Thus, the average temperature on November 15 would be for one day of a particular year, whereas the mean temperature for November 15 would be the average taken over several years for November 15.

It is important to note that the indexes found from weather records are for the air about 4-1/2 ft (1.4 m) above the ground; the value at the ground surface, which determines frost effects, is usually different, being generally smaller for freezing conditions and larger for thawing where surfaces are exposed to the sun. The surface index, which is the index determined for temperature immediately below the surface, is n times the air index, where n is the correction factor. Turf, moss, other vegetative cover, and snow will reduce the n value for temperatures at the surface of the mineral soil in relation to air temperatures and hence give less freeze or thaw penetration for the same air freezing or thawing index. n values for a variety of conditions are given in Table 1.

Table I

n-Factors for Freeze and Thaw
(ratio of surface index to air index)

Type of Surface (a)	For Freezing Conditions	For Thawing Conditions
Snow Surface (b)	1.0	-
Portland Cement Concrete	0.75	1.5
Bituminous Pavement	0.7	1.6 to 2+ (c)
Bare Soil	0.7	1.4 to 2+ (c)
Shaded Surface	0.9	1.0
Soil Under Turf Cover	0.5	0.8
Soil Under Trees and Low Ground Cover	0.3 (d)	0.4

(a) Surface exposed directly to sun and/or air without any overlying dust, soil, snow, ice, vegetation or shading except as noted otherwise, and with no building heat involved.

(b) Albedo only. Treat effect of snow depth on ground surface index separately.

(c) Use lowest value except in extremely high latitudes or at high elevations where a major proportion of summer heating is from solar radiation.

(d) Data from Fairbanks, Alaska, for a single season with normal snow cover permitted to accumulate.

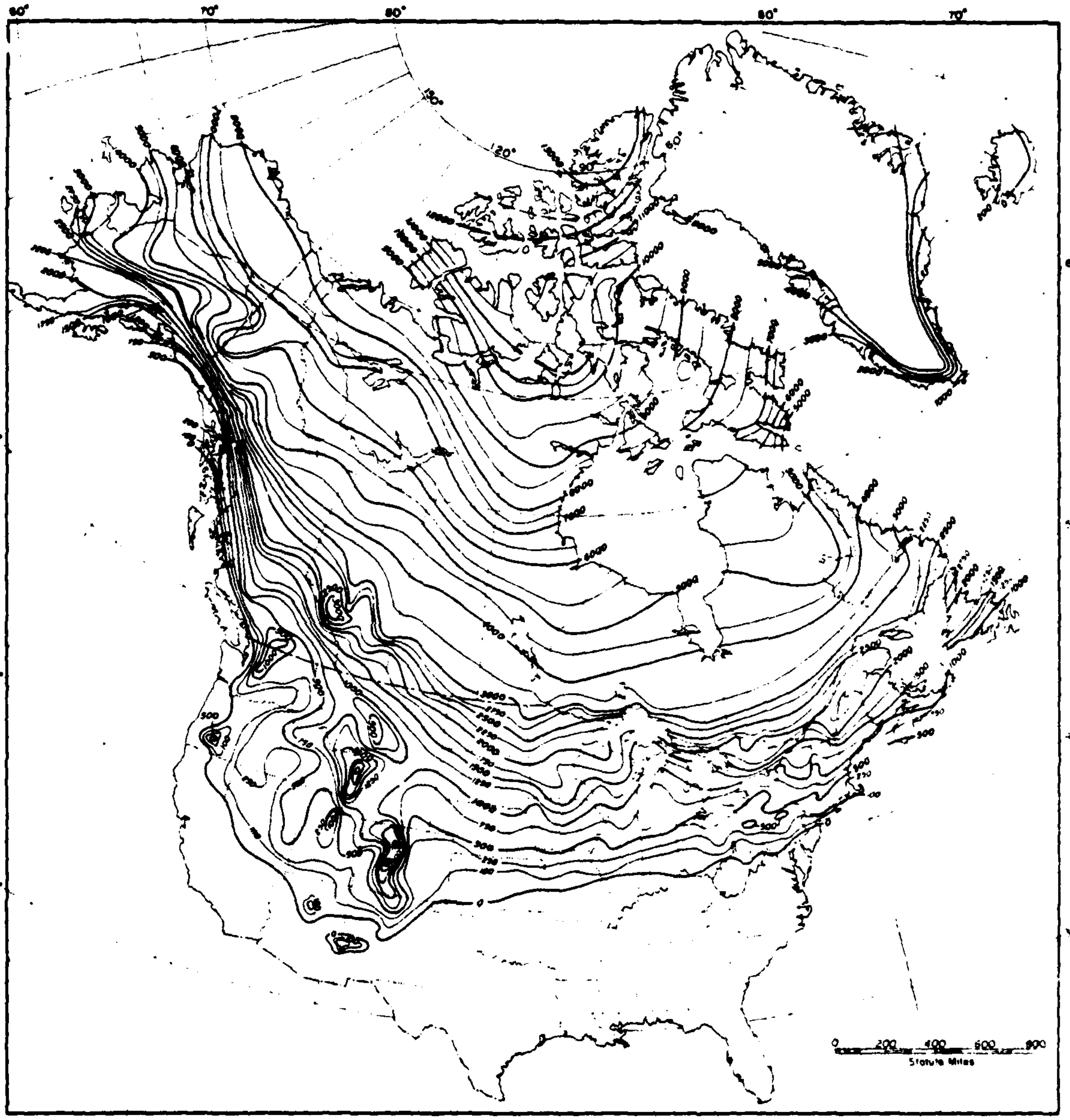
Figure 6 gives the approximate distribution of mean air-freezing and Figure 7 gives air-thawing intensities in North America.

3. FOUNDATION MATERIALS. The foundation design decisions may be critically affected by the foundation soil, ice and rock conditions.

a. Soils. The most important properties of soils affecting the performance of engineering structures under seasonal freeze-thaw action are their frost-heaving characteristics and their shear strength on thawing. Figure 8 summarizes relative frost-heaving qualities of soils, classified by the Unified Soil Classification System[1], when tested in the laboratory in an open system with unlimited availability of water and under standardized conditions. The values of rate of heave are comparative only, and should not be considered indicative of actual rates of heave which may occur under average field conditions.

The following criteria for frost-susceptible soils are used for pavement design purposes and may be considered for reference purposes:

> Most inorganic soils containing 3% or more of grains finer than 0.02 mm in diameter by weight are frost-susceptible. Gravels, well-graded sands, and silty sands, especially those approaching the theoretical maximum density curve, which contain 1-1/2 to 3% finer by weight than 0.02-mm size should be



NOTES

Mean air-freezing index values are cumulative seasonal degree-days below 32°F, computed on basis of mean monthly air temperature data.

The isolines of mean air-freezing index were drawn using data from approximately 800 weather stations. The map is offered as a guide only. It does not attempt to show local variations which may be considerable, particularly in mountainous areas.

The actual mean air-freezing index used should be computed for the specific project using temperature data from station nearest site

Figure 6. Distribution of mean air-freezing index values in North America (Fahrenheit).

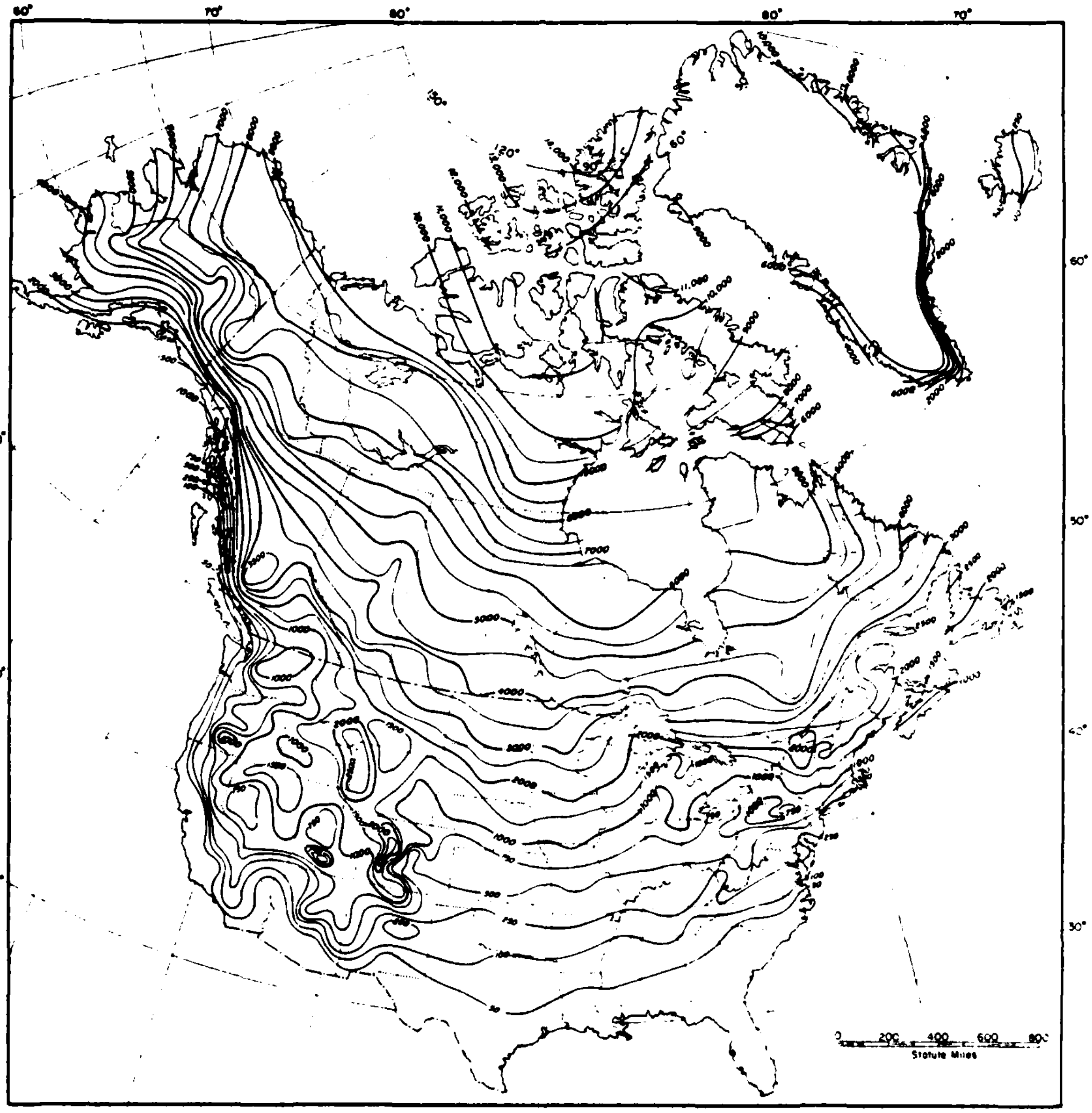
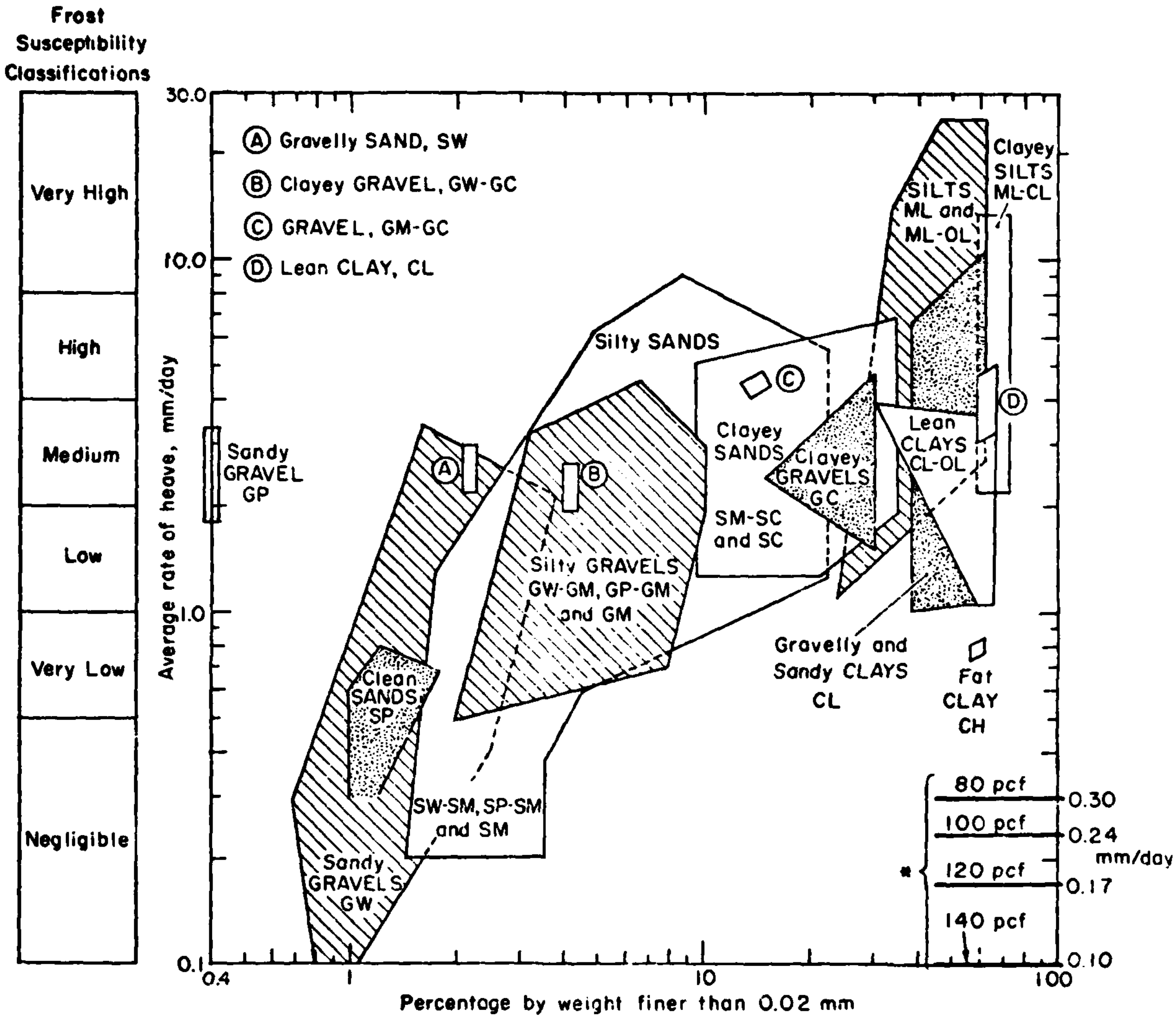


Figure 7. Distribution of mean air-thawing index values in North America (Fahrenheit).

considered as possibly frost-susceptible and should be subjected to a laboratory frost-susceptibility test to evaluate actual behavior during freezing. Uniform sandy soils may have as high as 10% of grains finer than 0.02 mm by weight without being frost-susceptible. However, their tendency to occur interbedded with other soils usually makes it impractical to consider them separately.



* Indicated heave rate due to expansion in volume, if all original water in 100% saturated specimen were frozen, with rate of frost penetration 0.25 inch per day.

Figure 8. Summary of average rate of heave vs. percentage of natural soil finer than 0.02 mm size. (29)

Heave potential at the lower limits of frost susceptibility determined by the above criteria is not zero, as may be seen from Figure 8, although it is generally low to negligible from the point of view of pavement applications. Applicability of these criteria to foundation design will vary, depending upon the nature and requirements of the particular construction.

Clean GW, GP, SW, and SP soils with a negligible precentage of material smaller than 0.02 mm are so completely nonheaving that foundation design on such materials is not usually governed by frost effects. If 100 percent saturated, it is possible for such soil to heave a small amount on freezing because of the expansion of water on changing to ice, provided this expansion cannot be compensated by ready escape of excess water through the soil immediately below the plane of freezing. However, if the expansion of the water which freezes can be balanced by movement of an equal volume of unfrozen water away from the freezing plane, there will be no expansion. Superficial fluffing of the surface of unconfined, relatively clean sands and gravels is frequently observed, but this effect is negligible when these materials are confined, as is the case in foundations. In fine-grained soils, ice segregation will occur upon freezing if water is available. Even in a closed system, there will be water migration toward the ice crystals as they form, causing ice segregation. The ice segregation is most commonly in the form of lenses and layers, oriented generally at right angles to the direction of heat flow, but it may also occur so uniformly distributed that it is often not readily apparent to the unaided eye; the surface heave is approximately equal to the total thickness of the ice layers. Pressure reduces heave, but heave forces normal to the freezing plane are very great, and may reach more than 10 tons per sq ft (para. 5). When fine-grained soils thaw, water tends to be released by melting of excess ice more rapidly than it can be drained away or redistributed in the thawed soil.

Permafrost soils cover the entire range of types from very coarse bouldery glacial drift to clays and organic soils. Strength properties of frozen soils are dependent on such variables as gradation, density, degree of saturation, ice content, unfrozen moisture content, temperature, dissolved solids, and rate of loading. Figure 9 shows typical compressive strength data for a frozen fine sand. Frozen soils characteristically exhibit creep at stresses as low as 5 to 10 percent of the rupture strength in rapid loading. Typical stress-creep relationships are shown in Figure 10.

b. Ice. In the annual frost zone, excess ice is formed by the common ice segregation process, although small amounts of ice may also originate from filling of shrinkage cracks; ice formations in this zone disappear each summer. Below the annual frost zone, excess ice in permafrost may have been formed by the same type of ice segregation process as above, may occur as vertical ice wedges formed by a horizontal contraction-expansion process, or may be "fossil ice" buried by landslides or other events. In permafrost areas, the possible effects of excess ice must be taken into account in the design if it exists in strata below the level of foundation support and thaw may reach the ice during the life of the structure. Although most common in fine-grained soils, substantial bodies of ground ice, more or less clear ice in the ground, are not uncommon in permanently frozen clean, granular deposits and bedrock.

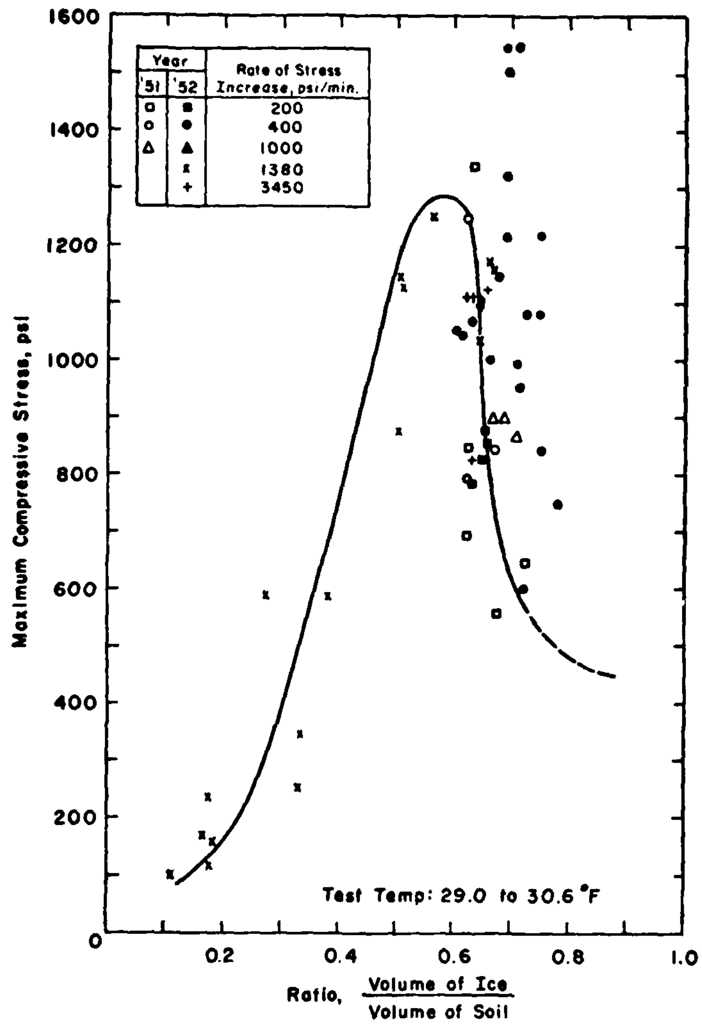


Figure 9. Compression strength vs. ice content, Manchester fine sand[18] (1953).

c. Rock. Bedrock subject to freezing temperatures should never be assumed problem-free in absence of positive subsurface information. In seasonal frost areas, mud seams in bedrock or concentrations of fines at or near the rock surface, in combination with the ability of fissures in the rock to supply large quantities of water for ice segregation, frequently cause severe frost heave. In permafrost areas, very substantial quantities of ice are often found in bedrock, occurring in fissures and cracks and along bedding planes.

4. WATER CONDITIONS. If a freezing soil has no access to free water beyond that contained in voids of the soil at and immediately below the plane of freezing, frost heave will necessarily be limited. However, if

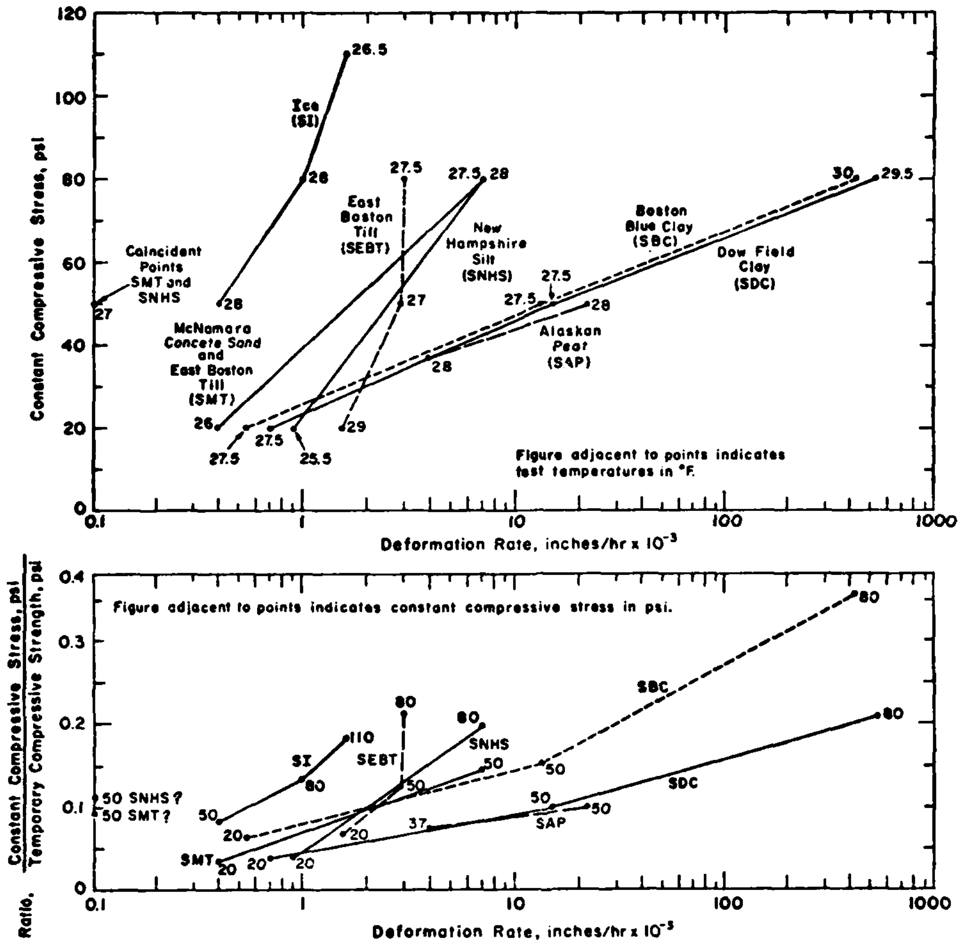


Figure 10. Plastic deformation of frozen soils under constant compressive stress.(17)

water drawn to the plane of freezing can be easily replenished, as from an aquifer layer or from a water table underlying at shallow depth, heave can be large. Water can migrate upward to the freezing plane with minimum difficulty from a water table within 5 ft (1.5 m) of the plane of freezing. Lowering of the water table to even great depth cannot be depended upon to eliminate frost heave, as the percentage of water that can be drained by gravity from most frost-susceptible soils is limited and may be negligible; the remaining water in the voids will still be available. Perched water tables can be as serious as true groundwater tables; care should be taken in explorations to detect all water tables.

In permafrost areas the supply of water available to feed growing ice lenses tends to be limited because of the presence of the underlying impermeable permafrost layer, usually at relatively shallow depths, and

maximum heave may thus be less than under otherwise similar conditions in seasonal frost areas. However, uplift forces on structures may be higher because of lower soil temperatures and consequent higher effective tangential adfreeze strength values.

The water content of soil exerts a substantial effect upon the depth of freeze or thaw penetration which will occur with a given surface freezing or thawing index. An increase in moisture content tends to reduce penetration by increasing the volumetric latent heat of fusion, as well as the volumetric specific heat capacity. While increase in moisture also increases thermal conductivity, the effect of latent heat of fusion tends to be predominant. TM 5-852-6(10) contains charts showing thermal conductivity relationships.

5. EFFECT OF SURCHARGE. For some engineering construction, complete prevention of frost heave is unnecessary and uneconomical. For most permanent structures, however, complete prevention is essential. Under favorable soil and foundation loading conditions, it may be possible to take advantage of the effect of surcharge to control heave. It has been demonstrated in laboratory and field experiments that the rate of frost heaving is decreased by increase of loading on the freezing plane and that frost heaving can be completely restrained if sufficient pressure is applied. Figure 11 shows results of a field experiment on a silt subgrade near Fairbanks, Alaska, in the form of a plot of seasonal heave versus subgrade frost penetration for various intensities of total stress at the freezing interface (which includes both weight of frozen

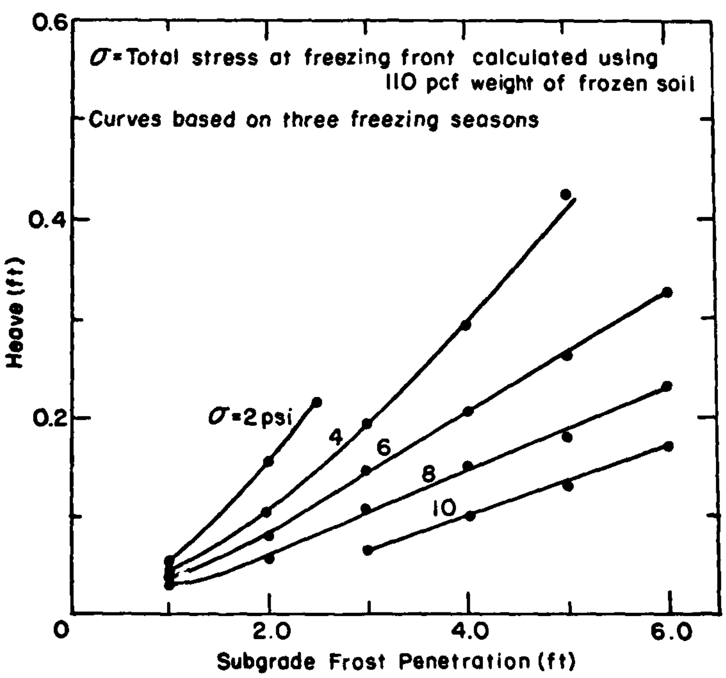


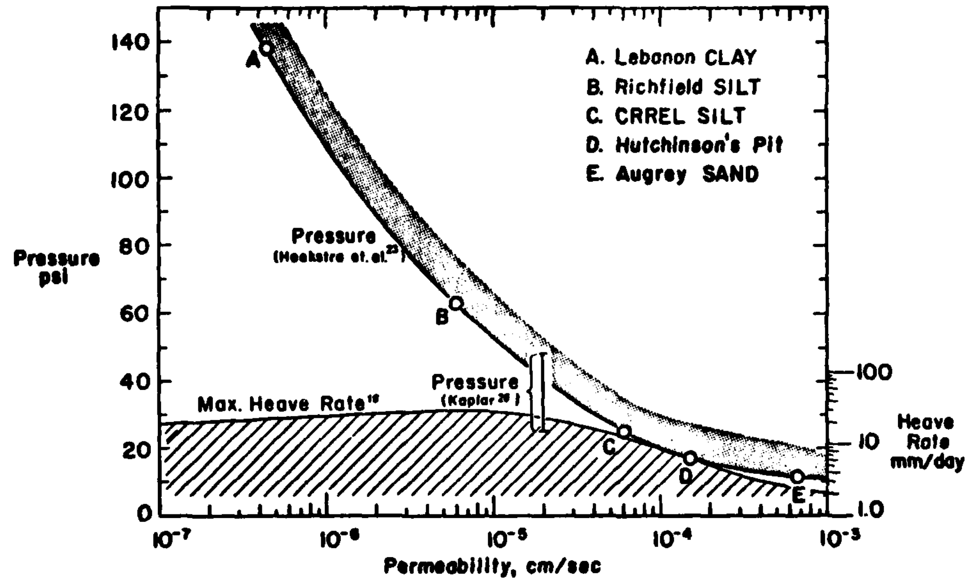
Figure 11. Heave vs. frost penetration for various total stresses, surcharge field experiment.(14)

soil and applied surcharge). These data show, for example, that for 5 ft (1.52 m) of seasonal subgrade frost penetration an increase in total stress at the freeze-thaw interface from 4 to 10 psi (2810 to 7030 kg/m^2) reduced frost heave from 0.4 to 0.15 ft (12 to 4.6 mm). No quantitative field-scale test data on clay-type subgrades are available, but it may be inferred from laboratory data and theory that clay subgrades will exhibit less rapid reduction of heave with increase in applied stress than is found for silt.

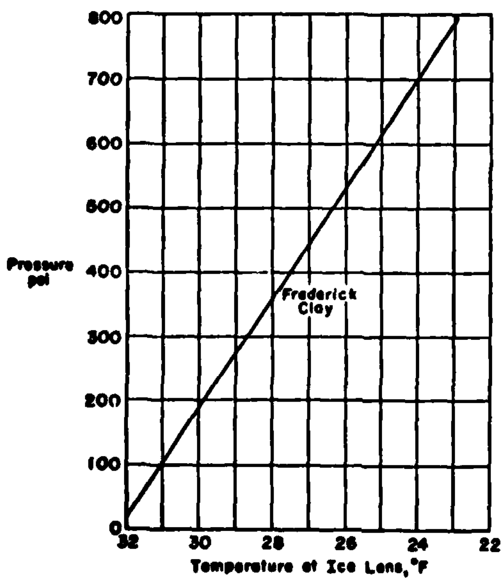
In laboratory freezing experiments, heaving pressures of the magnitudes shown by the pressure vs permeability data in Figure 12a and the pressure vs temperature data in Figure 12b have been measured under conditions of essentially complete restraint. Therefore, if foundation loadings at the freezing plane equal or exceed these pressures, heave will be prevented. However, potential frost uplift cannot be computed by simply applying the pressures of Figure 12 to the areas of direct foundation loadings, as frost heave acts on the base of a frozen slab of soil whose effective area may be much greater than the area of the structure foundation, as illustrated in Figure 13a. Also jacking forces acting on lateral surfaces may further complicate the situation, as illustrated in Figure 13b.

6. TYPE OF STRUCTURE. The type and uses of a structure affect the foundation design in frost areas as in other places. Wood frame structures have relatively high flexibility and capability for adjustment to differential foundation movements. Steel frame structures are more rigid than wood frame structures but more tolerant of movement than reinforced concrete and masonry, which require foundations providing complete freedom from cracking or distortion. Whether a structure is heated or not is also an important consideration if detrimental frost action or permafrost degradation are possibilities. Heat escape into the ground under a building can help to safeguard against foundation frost heave but would be detrimental if permafrost susceptible to thaw-settlement is present in the foundation. Aprons, loading docks, stairs and other exterior appurtenances must take into consideration the effects of frost heave and/or thaw subsidence in their design. Foundation requirements for an unheated warehouse or storage building may be quite different than those for a power plant. In the case of a structure which may be deactivated for a period, a decision must be made whether it will be designed for the frost conditions which will prevail when unheated or whether heat will be maintained during deactivation.

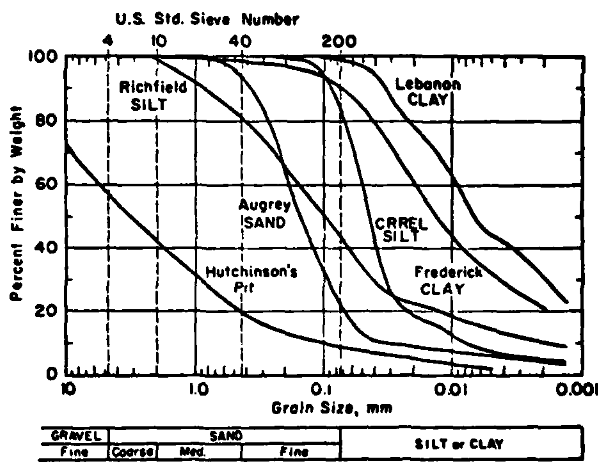
7. STRUCTURAL MATERIALS. In readily accessible seasonal frost areas, suitable construction materials are usually comparatively easy to obtain. In these areas, the effects of low temperatures on construction and the extent of special winter construction procedures required are in proportion to the freezing index. In remote areas of very intense seasonal freezing and in most permafrost areas, the situation is frequently difficult. Native materials are often scarce and not very suitable, and construction practices may differ because of transportation problems, equipment and labor available, and severe weather. In these areas, the relative cost and availability of all construction materials must be



a. Pressure and heave vs permeability.

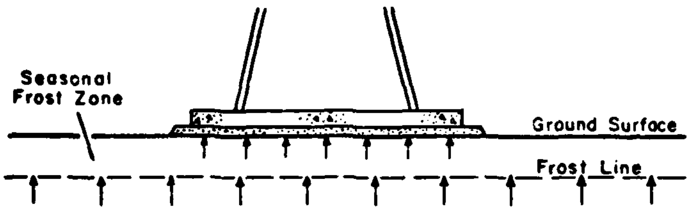


b. Temperature at growing ice lens vs maximum pressure developed at that depth.[32]

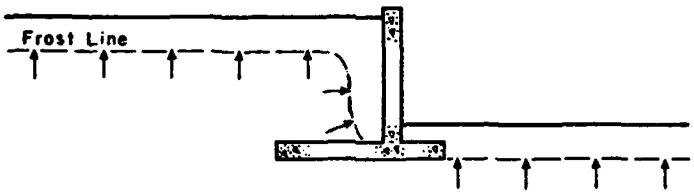


c. Grain size distribution of soils.

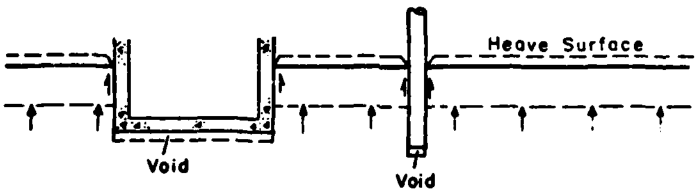
Figure 12. Maximum frost heave pressures.



a. Heaving of soil in seasonal frost zone causing direct upward thrust on overlying structural elements.



b. Freezing of frost-susceptible soil behind walls causing thrust perpendicular to freez-front.



c. Force at base of freezing interface tends to lift entire frozen slab, applying jacking forces to lateral surfaces of embedded structures, creating voids underneath. Structures may not return to original positions on thawing.

Figure 13. Frost action effects.

established early in the design; this will influence the selection of the foundation type to be used. Thermal characteristics of materials may be as important as mechanical properties.

a. Wood. Wood is a satisfactory and widely used material in cold regions because of such factors as availability, low thermal conductivity

and low weight. However, in many arctic and subarctic areas it may be as unavailable as any other construction material. In general, deterioration of wood under exposure to dry cold is very slow. However, for wood installed in the ground, untreated members may be destroyed at the ground line in only a few years even where the mean annual precipitation is low. Therefore, wood piles should be pressure treated, except that wood piles capped at the permafrost table need not be treated. Wood piles cannot be driven into permafrost by hammer; in some cases this may dictate the use of steel piles. Wooden structures require extra care to avoid the possibility of fire, which is a hazard of major concern in many cold regions.

b. Metals. As temperature is lowered, the hardness, yield strength, modulus of elasticity, and fatigue resistance of most metals and alloys increase. However, many of these same metals become embrittled at reduced temperatures and will shatter or fracture when subjected to stresses (especially due to impact) that would be allowable at normal temperatures. A small group of metals remains ductile at low temperature, including nickel, copper, aluminum, lead and silver. Several other metals such as magnesium, zinc and beryllium are brittle, with little ductility at room or slightly higher temperatures. Ferrous metals vary very widely in their behavior at low temperature. The brittleness of carbon steels increases with the carbon content (up to 0.25 percent), and higher carbon steels may be expected to be brittle. Almost all aluminum and titanium alloys can be used at low temperatures (except high strength aluminum, in which stress concentrations are likely to occur) and essentially all nickel and copper base alloys. Brass may shatter and is unsuitable for low temperature use. Based on years of experience with mild steels in structures throughout Alaska, it should not be necessary to use special alloys in structures and foundations when the number of fatigue cycles or dynamic stress level is low. However, the possibility of brittle fracture in either structural components or construction equipment under other conditions, such as during the construction phase, must be kept in mind.

Examination of steel pipe and H-piles installed for periods of 8-11 years at the USACRREL Alaska Project Office at Fairbanks, Alaska, showed that the length of pile embedded in permafrost was unaffected by corrosion and only insignificant effects were observed in the annual thaw zone[39,40]. Thus, protective coating of steel in the annual thaw zone in permafrost areas is optional; it may be needed only under special local conditions. Paint or similar protective coatings reduce the potential tangential shear which can be developed between frozen soil and pile surfaces and care must be taken that such materials are not applied below the level of the permafrost table which will exist after construction. Protective coatings which are brittle or which have linear shrinkage coefficients widely different from the protected material should be avoided. Galvanizing has shown cracking and spalling when exposed to cold in Arctic areas if the coating was too thick.

In very cold areas, the high coefficients of thermal conductivity of metals must be considered in relation to heat conduction problems and possible thawing of permafrost.

c. Concrete and Masonry. When properly employed, good quality concrete is very useful in cold regions construction and presents no fire hazard. Freezing-and-thawing cycles in completely cured good quality air-entrained concrete are not generally harmful, but if the concrete is below standard or if especially adverse factors exist at the time of the freeze-thaw cycles the effects may be serious. Concrete which will be exposed to frost action should have (1) durable aggregates, (2) 4 to 7 percent entrained air depending on aggregate gradation, (3) proper consistency for good placement without segregation, (4) adequate curing, and (5) best possible drainage afterwards. Concrete which is saturated prior to freezing tends to be more susceptible to freeze-thaw damage. For large or critically exposed structures, investigation and testing of available aggregates including freeze-thaw testing, petrographic analysis, and detailed mix-design studies are justified. Frost action is less detrimental in areas which are so cold that materials remain frozen throughout the winter than in warmer areas where frequent freeze-thaw cycles occur during the winter months.

Because concrete needs to be warm during mixing, placement and curing and because it generates heat internally for a considerable time, difficulties arise when concrete is to be placed on or near permafrost (see TM 5-852-6(10) for computations on heat of hydration and its effects). If the concrete section is thin and the temperature of the ground well below freezing, the concrete may not set and harden properly and may even freeze; if the concrete section is massive, its heat of hydration will be sufficient to thaw the ground for some depth, and this may result in settlement and loss of design grades, formation of voids, and cracking. When concrete has to be placed on or close to permafrost, special pads of gravel, wood or rigid insulating materials are used between the frozen ground and the concrete course to protect the frozen ground from thawing and to aid in retention of heat by the concrete. Precast pads on a gravel course may also be used. Mechanical refrigeration has been used on important work to maintain the 32°F (0°C) point on the temperature gradient at the proper position between the underlying permafrost and the overlying new concrete during cure. It should be noted that concrete hardens very slowly at low temperatures but shrinks badly if cured quickly by too much heat. When the temperature of the concrete is below 32°F (0°C), the concrete will not gain strength.

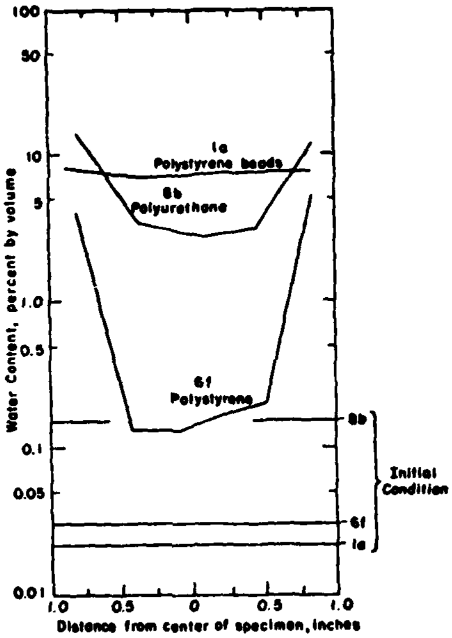
If a possibility of frost heave or thaw-settlement distortion exists, concrete block or brick masonry construction should be avoided because of its poor ability to tolerate differential movements. Bricks which will be in contact with frozen soil should be of SW (ASTM) grade or equivalent. The mortar must be durable and moisture resistant.

d. Thermal Insulating Materials. Thermal conductivity values of construction materials are usually given for the dry condition. However, the values are greatly affected by moisture. Most materials lose much of their insulation value if they become wet, and most conventional insulating materials absorb water easily. Figure 14 shows measured moisture distribution in some kinds of insulation board under various conditions. Additional information is given in TM 5-852-4[8]. To prolong effective-

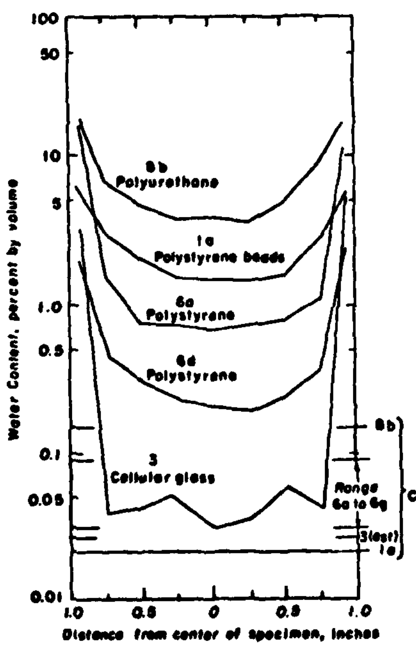
No.	Board Composition	Approx. Density ρ pcf
1a	Fused expanded polystyrene beads	0.9
1b	" " " "	1.6
2	Glass fibers, sandwich	9.5
3	Cellular glass	9.2
4	Corkboard	15.0
5	Perlite beads with organic fiber	10.7
6a	Polystyrene, extruded	1.7
6b	" "	1.9
6c	" "	2.0
6d	" "	2.5
6e	" "	2.5
6f	" "	3.1
6g	" "	3.6
7	Asbestos with binder	14.8
8a	Polyurethane	1.9
8b	"	2.2

All insulations were plant manufactured.

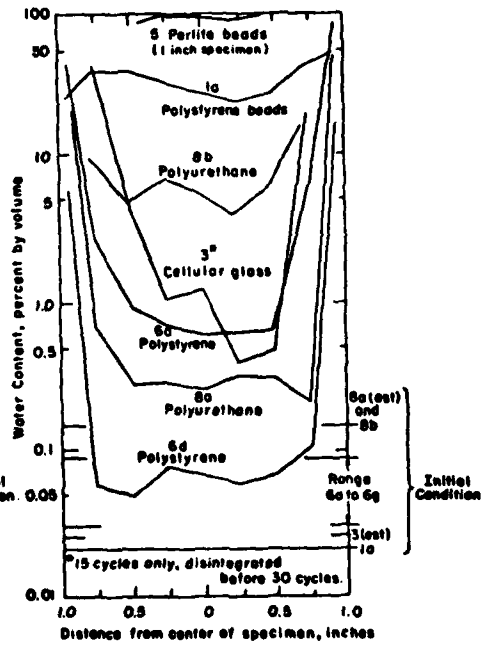
a. Index for insulating materials.



b. After 18 months in water.



c. After 34 months embedment in moist silt.



d. After 30 freeze-thaw cycles (over approx. 30-day period) with specimen immersed in water.

Figure 14. Internal moisture distribution in insulation board under different test conditions.[30]

ness of insulation in foundation construction, it should wherever possible be placed in positions offering minimum exposure to moisture and to moist freeze-thaw conditions. Edge insulation may be required for floor slabs on grade to prevent excessive heat loss, maintain a comfortable floor temperature, and prevent condensation on the floor surface adjacent to exterior walls[38,46]. Insulation material for foundation use should exhibit minimum absorption, and allowance should be made in the design for the degree of insulation impairment from moisture which is expected over the life of the facility. Details of manufacture may significantly affect moisture absorption rates. Cellular glass has been extensively used in underground situations but both laboratory and field experiments show that it progressively deteriorates under wet freeze-thaw conditions. Cell concrete, vermiculite concrete and other forms of lightweight concrete which will gradually absorb substantial amounts of moisture when placed underground are unsuitable as insulating materials in foundations. Insulation slows rate of heat flow but it cannot completely prevent heat flow. When thick pads of non-frost-susceptible granular materials are used under structures, they provide thermal buffers or heat sinks in which freezing and thawing occur with minimum detrimental effects; if freeze and thaw should penetrate somewhat into the soil below such a granular pad, the pad helps to reduce the magnitude of frost heave by its weight and to distribute effects of any nonuniform changes which occur in the ground.

SITE INVESTIGATIONS

1. GENERAL. In addition to normal temperate zone site investigations and data, design of foundations in areas of significant frost penetration requires special studies and data because of factors introduced by the special frost-related site conditions. Methods of terrain evaluation for arctic and subarctic areas are given in TM 5-852-8[12]. Detailed site investigation procedures applicable for arctic and subarctic areas are described in TM 5-852-2[6] and in Section 3 of TM 5-852-4[8] and may be adapted or reduced in scope, as appropriate, in areas of less severe winter freezing.

2. REMOTE SENSING AND GEOPHYSICAL INVESTIGATIONS. These techniques are particularly valuable in selection of the specific site location, when a choice is possible. They can give clues to subsurface frozen ground conditions because of effects of ground freezing upon such factors as vegetation, land wastage, and soil and rock electrical and acoustical properties.

3. DIRECT SITE INVESTIGATIONS. The number and extent of direct site explorations should be sufficient to reveal in detail the occurrence and extent of frozen strata, permafrost and excess ice including ice wedges, moisture contents and ground water, temperature conditions in the ground, and the characteristics and properties of frozen materials and unfrozen soil and rock.

The need for investigation of bedrock requires special emphasis. In seasonal frost areas, bedrock is frequently a source of severe frost heave

because of mud seams in the rock or concentrations of fines at or near the rock surface, in combination with the ability of fissures in the rock to supply large quantities of water for ice segregation. In permafrost areas, permanently frozen bedrock often contains large masses of ice which would produce substantial settlement on thaw. Bedrock in permafrost areas should be drilled to obtain undisturbed frozen cores whenever ice inclusions could affect the foundation design or performance.

In areas of discontinuous permafrost, sites require especially careful exploration and many problems can be avoided by a proper site selection. As an example, the moving of a site 50 to 100 ft (15 to 30 m) from its planned position may place a structure entirely on or entirely off permafrost, in either case simplifying the foundation design. A location partly on and partly off permafrost might involve an exceptionally difficult or costly foundation design.

Because frozen soils may have compressive strengths as great as that of a lean concrete and because ice in the ground may be melted by conventional drilling methods, special techniques are frequently required for subsurface exploration in frozen materials. Core drilling using refrigerated drilling fluid to prevent melting of ice in the cores provides specimens which are nearly completely undisturbed and can be subjected to the widest range of laboratory tests[25]. By this procedure, soils containing particles up to boulder size and bedrock can be sampled and ice formations can be inspected and measured. Drive sampling is feasible in frozen fine-grained soils above about 25°F (-4°C) and is often considerably simpler, cheaper and faster[33]. Samples obtained by this procedure are somewhat disturbed, but they still permit accurate ice and moisture content determinations. Test pits are very useful in many situations. For frozen soils which do not contain very many cobbles and boulders, truck-mounted power augers using tungsten carbide cutting teeth will provide excellent service where classification, gradation, and rough ice-content information will be sufficient. In both seasonal frost and permafrost areas, a saturated soil condition is common in the upper layers of soil during the thaw season, so long as there is frozen, impervious soil still underlying. Explorations attempted during the that season are handicapped and normally require that borings be cased through the thawed layer. In permafrost areas, it is frequently found more desirable to carry out explorations during the colder part of the year, when the annual frost zone is frozen, than during the summer.

In subsurface explorations which encounter frozen soil, it is important that the boundaries of frozen and thawed zones and the amount and mode of ice occurrence be recorded. Materials encountered should be identified in accordance with the Unified Soil Classification System, including the special system for frozen soils.[1]

In seasonal frost areas, the most essential site data beyond those needed for non-frost foundation design are the design freezing index and the soil frost susceptibility characteristics. In permafrost areas the data requirements are considerably more complex, as described in

TM 5-852-4[8]; there, determination of the susceptibility of the foundation materials to settlement on thaw and of the subsurface temperatures and thermal regime will usually be the most critical special requirements. Ground temperatures are measured most commonly with copper-constantan thermocouples or with thermistors.

Special site investigations such as installation and testing of test piles or thaw-settlement tests may be required. Assessment of the excavation characteristics of frozen materials may also be a key factor in planning and design.

4. SITE TECHNICAL DATA. Foundation design will require technical information in some or all of the following categories:

- Physiography and geology
- Climate
- Subsurface thermal regime
- Identification and classification of foundation materials
- Density and ice content
- Thaw consolidation and settlement
- Thermal properties of foundation materials
- Ground water
- Frost susceptibility of soils
- Frost heave field observations
- Creep and solifluction
- Other data

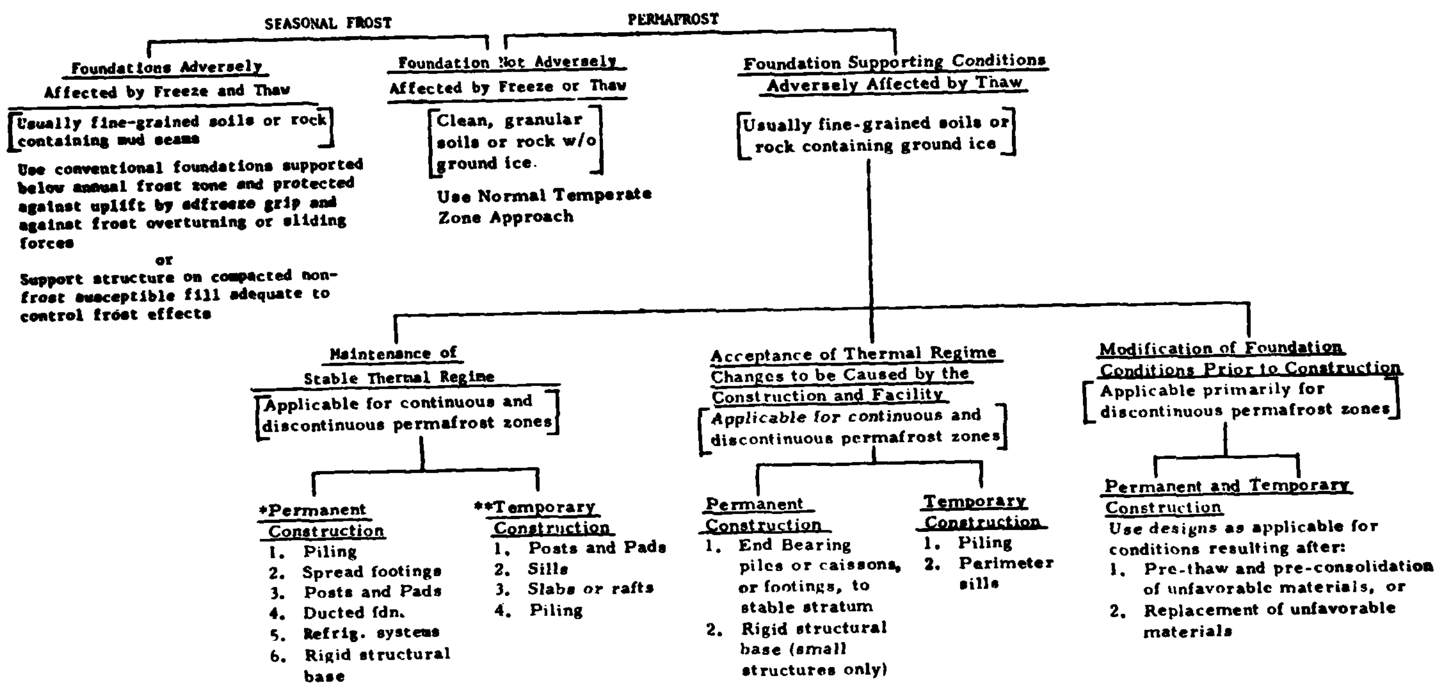
Guidance concerning each of the above data categories is presented in TM 5-852-4[8].

FOUNDATION DESIGN

1. SELECTION OF FOUNDATION TYPE. Only sufficient discussion of the relationships between foundation conditions and design decisions is given below to indicate the general nature of the problems and solutions. Greater detail is given in TM 5-852-4[8].

a. Foundations in Seasonal Frost Areas. When foundation materials within the maximum depth of seasonal frost penetration consist of clean sands and gravels or other non-frost-susceptible materials which do not develop frost heave or thrust, or thaw weakening, design in seasonal frost areas may be the same as for non-frost regions, using conventional foundations, as indicated in Figure 15. Effect of the frost penetration on related engineering aspects such as surface and subsurface drainage or underground utilities may need special consideration. Thorough investigation should be made to confirm the non-frost-susceptibility of subgrade soils prior to design for this condition.

When foundation materials within the annual frost zone are frost-susceptible, seasonal frost heave and settlement of these materials may occur. In order for ice segregation and frost heave to develop, freezing



As defined in Construction Criteria Manual, DOD4270.1-M, March 1968:

*Permanent Construction - Construction incorporating the type and quality of materials and equipment, and details and methods of construction, which results in a building or facility suitable to serve a specific purpose over a minimum life expectancy of 25 years with normal maintenance.

**Temporary Construction - Construction incorporating the type and quality of materials and equipment, and details and methods of construction, which results in a building or facility suitable to provide minimum accommodations at low first cost to serve a specific purpose for a short period of time, 5 years or less, in which the degree of maintenance is not a primary design consideration.

Figure 15. Design alternatives.

temperatures must penetrate into the ground, soil must be frost-susceptible, and adequate moisture must be available. The magnitude of seasonal heaving is dependent on such factors as rate and duration of frost penetration, soil type and effective pore size, surcharge and degree of moisture availability. Frost heave in a freezing season may reach a foot or more in silts and some clays if there is an unlimited supply of moisture available. The frost heave may lift foundations together with the structures they support, commonly differentially, with a variety of possible consequences. The frost pressure acts perpendicular to the freezing front within the ground as illustrated in Figure 13, and behind a vertical retaining wall the horizontal frost thrust may fracture or tilt the wall. Force may also be transmitted to a foundation by adfreeze grip as illustrated in Figure 13c.

When thaw occurs, the ice within the frost-heaved soil is changed to water and escapes to the ground surface or into surrounding soil, allowing overlying materials and structures to settle. If the water is released by thaw more rapidly than it can be drained away or redistributed, substantial loss in soil strength occurs. In seasonal frost areas, a heaved foundation may or may not return to its before-heave elevation. Friction on lateral surfaces or intrusion of softened soil into void space below the heaved foundation members may prevent full return. Successive winter seasons may produce progressive upward movement.

Therefore, when the soils within the maximum depth of seasonal frost penetration are frost-susceptible, foundations in seasonal frost areas should be supported below the annual frost zone, using conventional foundation elements protected against uplift caused by adfreeze grip and against frost overturning or sliding forces, or the structure should be placed on compacted non-frost-susceptible fill designed to control frost effects (Figure 15).

b. Foundations in Permafrost Areas. Design in permafrost areas must cope with both the annual frost zone phenomena described in paragraph a above and those peculiar to permafrost. The subject is covered in greater detail in TM 5-852-4[8].

(1) Permafrost Foundations not Adversely Affected by Thaw. Whenever possible, structures in permafrost areas should be located on clean, non-frost-susceptible sand or gravel deposits or rock which are free of ground ice or of excess interstitial ice which would make the foundation susceptible to settlement on thaw. Such sites are ideal and should be sought whenever possible. As indicated in Figure 15, foundation design under these conditions can be basically identical with temperate zone practices. This is true even though the materials are frozen below the foundation level, as has been demonstrated in Corps of Engineers construction in interior Alaska. When conventional temperate zone designs are used on such materials, heat from the structure will gradually thaw the underlying materials to progressively greater depths over an indefinite period of years. In five years, for example, thaw may reach a depth of about 40 ft (122 m). However, if the foundation materials are not susceptible to settlement on thaw, there will be no

effects on the structure from such thaw. Although surface geological formations will often indicate the presence of ice lenses or wedges, the absence of such evidence should not be taken to indicate that no excess ice is present. Conclusive evidence from carefully performed subsurface explorations and analysis of geological history is required.

Some frozen, clean, non-frost-susceptible sand and gravel deposits, though free of excess ice, may have sufficiently low relative densities, particularly in the upper 30 ft (9 m), so that settlement after thawing may be excessive for the kinds of facilities to be placed upon them. The possibility of consolidation of excessively loose materials by earthquake or other shock action after thawing should also be considered. When pertinent, such possibilities should be evaluated in the course of the site investigations. Procedures are described in TM 5-852-4[8]. The relative densities of sand and gravel formations can be determined by careful sampling operations and laboratory tests. The in-situ granular materials may be steam thawed followed by bearing capacity tests such as plate-bearing tests at the level of the proposed footings in test excavations. If the foundation is satisfactory except for looseness, prethawing and preconsolidation by dynamite shock or vibration may be necessary; however, experience to date in Alaska and Greenland indicates that such measures are usually unnecessary in clean, granular, non-frost-susceptible materials free of excess ice, except possibly for foundations subject to strong, prolonged vibration as under a diesel generator power station.

Caution must be exercised not to be misled by a mere cap of satisfactory sands and gravels over materials which contain segregated ice and which would in time be reached by thaw. On the other hand, when undesirable materials form a relatively shallow covering over clean sands and gravels, design may be greatly simplified by excavation of the poor materials and replacement with compacted sand or gravel, even though as much as 20 ft (6.1 m) of excavation and replacement may be necessary.

(2) <u>Permafrost Foundations Adversely Affected by Thaw</u>.

When permafrost foundation materials containing excess ice are thawed, the consequences may include differential settlement, development of water-filled surface depressions which serve to intensify thaw, loss of strength of frost-loosened foundation materials under excess moisture conditions, slope instability, development of underground uncontrolled drainage channels in materials susceptible to bridging or piping, and other detrimental effects. Often, the results may be catastrophic.

For permafrost soils and rock containing excess ice, design should consider the following alternatives, as indicated in Figure 15.

(a) <u>Maintenance of Stable Thermal Regime</u>. Under this design approach, which is applicable for both continuous and discontinuous permafrost zones, foundation materials thawed in summer are completely refrozen in the following winter; progressive annual lowering of the

permafrost table, thawing of ice in the ground, and settlement are prevented; and permafrost temperatures are not allowed to exceed limits for safe foundation support. Usually, maintenance of the thermal regime which existed before construction is sought, although some disturbance during construction is inevitable. Foundation cooling is normally achieved by circulation of cold winter air. Self-refrigerated piles may be used to assist cooling. Mechanical types of refrigeration are employed more rarely. This design approach is by far the most commonly used and most acceptable method.

(b) Acceptance of Thermal Regime Changes. Under this design approach, which is applicable for both continuous and discontinuous permafrost zones, settlement and other consequences of permafrost degradation caused by the construction and use of the facility are anticipated and accepted or allowed for. This approach is usually only used for temporary structures such as very temporary construction camp buildings. Possibility of unacceptable environmental impact must be considered.

(c) Modification of Foundation Conditions Prior to Construction. Under this design approach, which is applicable almost solely in the discontinuous or marginal permafrost areas, the ultimate effects of the construction on the thermal regime and permafrost are estimated and brought about in advance of the construction. Where conditions are favorable, this method may obviate the need for special foundation structural design, although the requisite conditions for successfully employing this technique occur somewhat rarely.

The above alternative approaches are discussed in TM 5-852-4[8]. Choice of the specific foundation type from among those indicated in Figure 15 can be made on the basis of cost and performance requirements after development of details to the degree needed for resolution.

2. FOUNDATION FREEZE AND THAW AND TECHNIQUES FOR CONTROL. Detailed guidance for foundation thermal computations and for methods of controlling freeze and thaw penetrations is presented in TM 5-852-4 and 6[8,10].

a. Design Depth of Ordinary Frost Penetration. For average permanent structures, the depth of frost penetration assumed for design, for situations not affected by heat from a structure, should be that which will occur in the coldest year in 30. For structures of a temporary nature or otherwise tolerant of some foundation movement, the depth of frost penetration in the coldest year in 10 or even that in the mean winter may be used, as may be most applicable. The design depth should preferably be based on actual measurements, or on computations if measurements are not available. When measurements are available, they will nearly always need to be adjusted by computations to the equivalent of the freezing index selected as the basis for design, as measurements will seldom be available for a winter having a severity equivalent to that value.

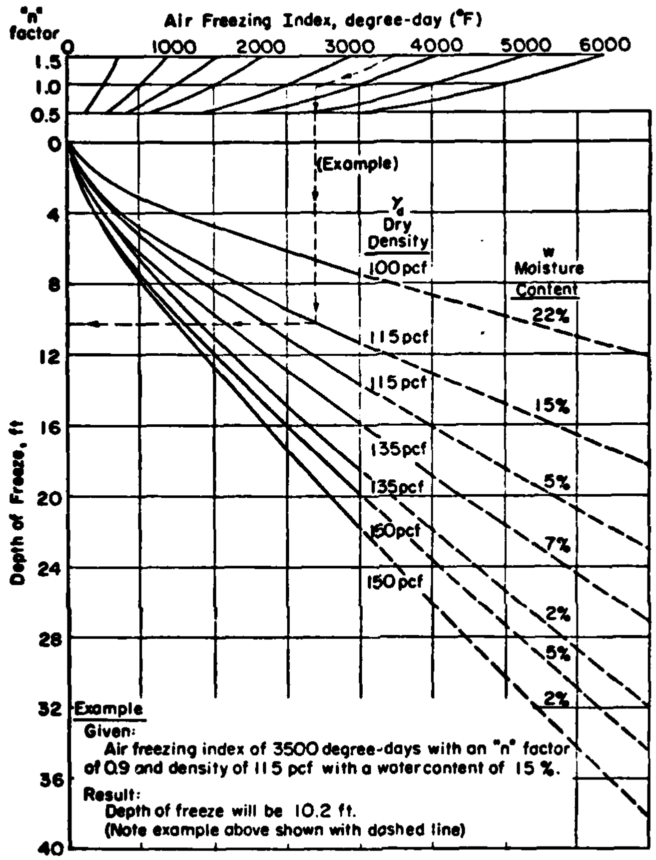
The frost penetration can be computed using the design freezing index and the detailed guidance given in TM 5-852-6[10]. For paved areas kept free of snow, approximate depths of frost penetration may be estimated from TM 5-818-2[3] or TM 852-3[7], entering the appropriate chart with air freezing index directly. Approximate values of frost penetration may also be estimated from Figure 16a for homogeneous soils of the density and moisture content ranges there represented.

For average conditions, the air freezing index can be converted to surface index by multiplying it by the appropriate n-factor from Table I, or Figure 16a can be entered directly with n and the air freezing index.

For given soil conditions, the greatest depths of frost penetration will be for paved outdoor areas kept cleared of snow and shaded from the sun. For heated buildings, the heat flowing outward from the foundation tends to modify frost penetration next to the foundation wall. Insulation or furring out of basement or perimeter walls will change heat flow. The effect of snow cover should usually be disregarded for design purposes, as snowfall may be very small or negligible in the years when temperatures are coldest. Turf, muskeg, and other vegetative covers help substantially to reduce frost penetration, but natural cover of this type is usually destroyed near foundations during construction. Some additional guidance on effects of surface conditions is contained in TM 5-852-6[10].

In the more developed parts of the cold regions, the building codes of most cities specify minimum footing depths, based on many years of local experience; these depths are invariably less than the maximum observed frost penetrations. The code values should not be assumed to represent actual frost penetration depths. Such local code values have been selected to give generally suitable results for the <u>types of construction, soil moisture, density, and surface cover conditions, severity of freezing conditions, and building heating conditions</u> which are common in the area. Unfortunately, the code values may be inadequate or inapplicable under conditions which differ from those assumed in formulating the code, especially for unheated facilities, insulated foundations, or especially cold winters. Building codes in the Middle and North Atlantic States and Canada frequently specify minimum footing depths in the range of 3 to 5 ft (0.9 to 1.5 m). If frost penetrations of this order of magnitude occur with fine silt and clay type soils, 30 to 100 percent greater frost penetration may occur in well-drained gravels under the same conditions. With good soil data and a knowledge of local conditions, computed values for ordinary frost penetration, unaffected by building heat, may be expected to be adequately reliable, even though the freezing index may have to be estimated from weather data from nearby stations. In remote areas, measured frost depths may be entirely unavailable.

b. <u>Design Depth of Ordinary Thaw Penetration</u>. Seasonal thaw penetration in permafrost areas of the Arctic and Subarctic typically begins in May or June and reaches maximum depth in the ground in the period July -

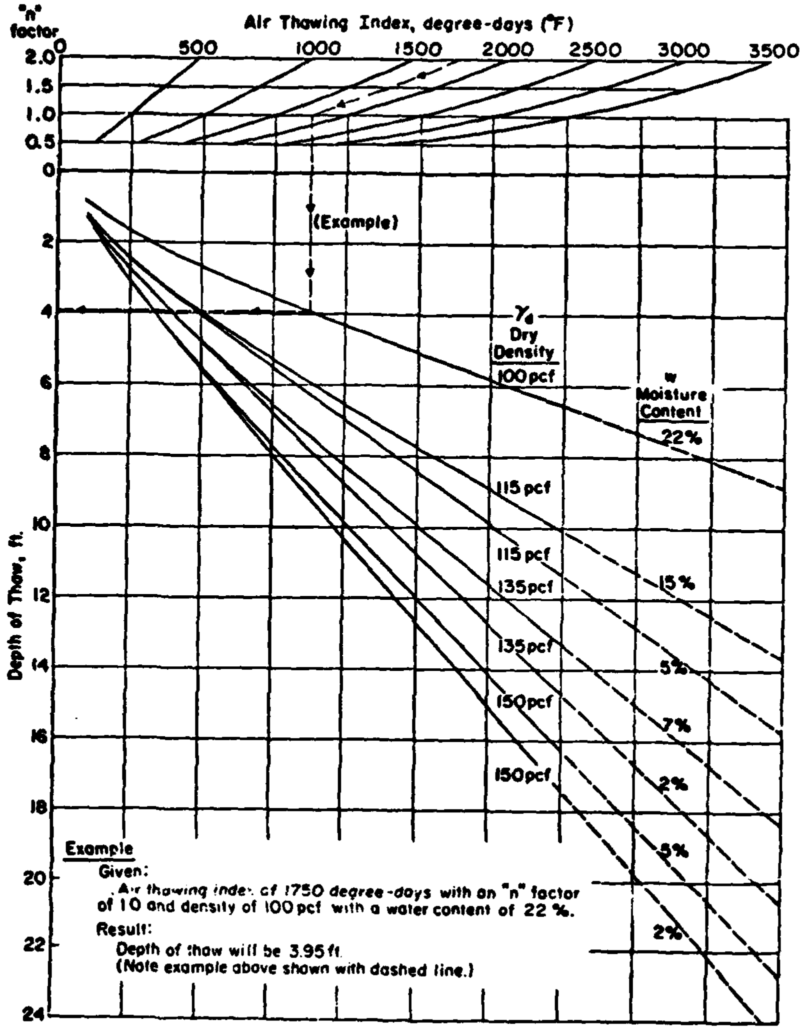


NOTES:

1. In calculations for the curves, thermal conductivities for frozen and thawed states have been averaged together. Because the actual effective thermal conductivity may not be equal to this average value during either freezing or thawing, precise agreement between measured and predicted values should not be anticipated. However, deviations due to this approximation should not exceed those arising from other causes.
2. Curves developed from calculations based on procedures in TM 5-852-6

a. Air freezing index vs depth of freeze.

Figure 16. Approximate depth of freeze or thaw vs. air freezing or thawing index and "n" factor for various homogeneous soils.



b. Air thawing index vs depth of thaw.

Figure 16 (cont'd). Approximate depth of freeze or thaw vs air freezing or thawing index and "n" factor for various homogeneous soils.

September. Under paved areas exposed to sunshine, particularly black bituminous pavements, seasonal thaw penetrations in high density, extremely well-drained granular materials may be substantial and in marginal permafrost areas may reach as much as 20 ft (6.1 m). Thaw depths under undisturbed natural surfaces reach values ranging from a few inches to several feet. Estimates of seasonal thaw penetration should be established on the same statistical and measurement bases as outlined in para. a above for seasonal frost penetration. The air thawing index can be converted to surface thawing index by multiplying it by the appropriate thawing-conditions n-factor from Table I. The

thaw penetration can then be computed using the detailed guidance given in TM 5-852-6[10]. Approximate values of thaw penetration may also be estimated from Figure 16b for homogeneous soils of the density and moisture content ranges there represented.

Degradation of permafrost will result if average annual depth of thaw penetration exceeds average depth of frost penetration.

c. Thaw or Freeze Beneath Structures. Any change from natural conditions which results in a warming of the ground beneath a structure can result in progressive lowering of the permafrost table over a period of years. Heat flow from a structure into underlying ground containing permafrost can only be ignored as a factor in the long term structural stability when the nature of the permafrost is such that no settlement or other adverse effects will result. The source of heat may be not only the building heat but also solar radiation, underground utilities, surface water and ground water flow.

Figure 17 shows an idealized diagram of the effect of size on total depth of thaw and rate of thaw under a heated structure placed directly on frozen material. The larger the structure the larger the potential

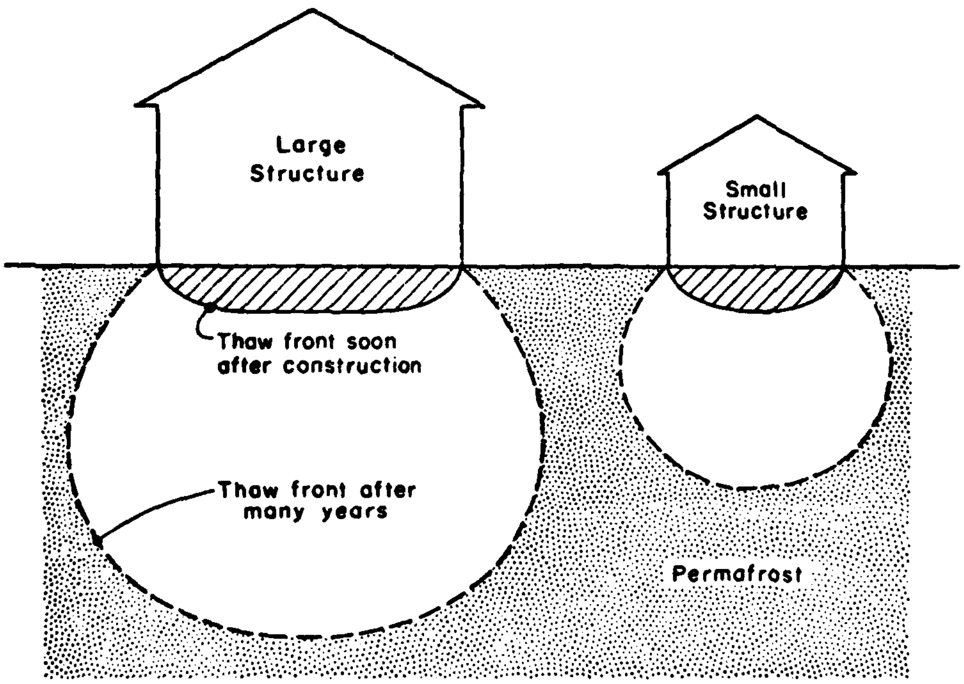
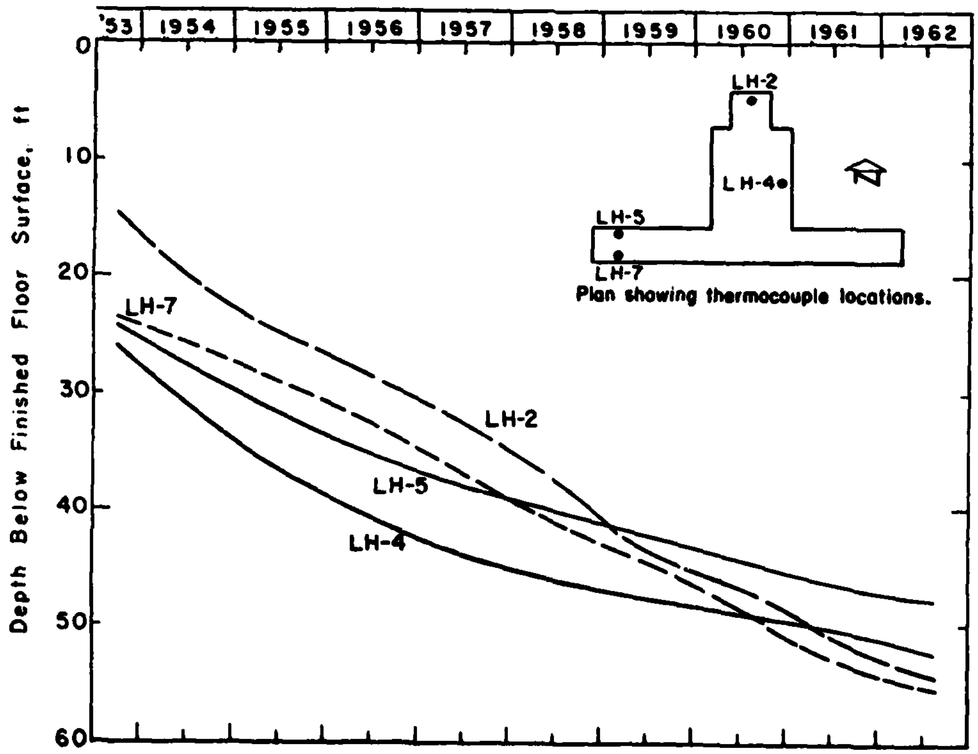


Figure 17. Effect of heated structure size on depth and rate of thaw.

ultimate depth of thaw, although the initial rate of thaw may be the same under the centers of both structures. Requirements for control of heat flow from the structure into the ground, for foundation complexity and refinement, and for maintenance may vary widely depending on subsurface conditions, the size of the structure, and whether the structure is temporary or permanent. TM-852-4 and -6(8,10) provide guidance on procedures for estimating the depth of thaw under a heated building with time.

Figure 18 shows measured degradation of permafrost under a heated 5-story reinforced concrete structure at Fairbanks, Alaska. Because the clean, granular permafrost foundation soils did not contain ice in such form as to cause significant settlement on thawing, the permafrost degradation had no adverse structural effects.



Foundation

Perimeter wall footing and interior spread footings. Uninsulated basement floor 3 to 6 ft below ground surface.

Soil types

Sandy gravels to silty sands.

Figure 18. Degradation of permafrost under 5-story reinforced concrete structure, Fairbanks, Alaska.

The most widely employed, effective and economical means of maintaining a stable thermal regime under a heated structure, without degradation of permafrost, is by use of a ventilated foundation. Under this scheme, provision is made for circulation of cold winter air between the insulated floor and the underlying ground. The air circulation freezes back the upper layers of soil which were thawed in the preceding summer, and progressive lowering of the permafrost table is prevented. In discontinuous permafrost areas, the annual freeze-back is more difficult to achieve than in colder climates. This may limit the feasible width of building for a given type of ventilated foundation design and may require use of stacks or chimneys to induce increased circulation and/or increased insulation thickness.

The simplest way of providing foundation ventilation is by providing an open space under the entire building, with the structure supported on footings as in Figure 19 or piling as in Figure 20. The minimum height of crawl space is usually 30 to 36 inches (76 to 91 cm). For heavier floor loadings, ventilation ducts below the insulated floor such as shown in Figure 21 and 22 may be used. Experience has shown that ventilated foundations should be so elevated, sloped, oriented and configured as to minimize possibilities of accumulation of water, snow, ice or soil in the ducts. Guidance in thermal analysis of ventilated foundations, including estimation of depths of summer thaw in underlying materials and design to assure winter refreezing, is given in TM 5-852-4 and -6[8,10].

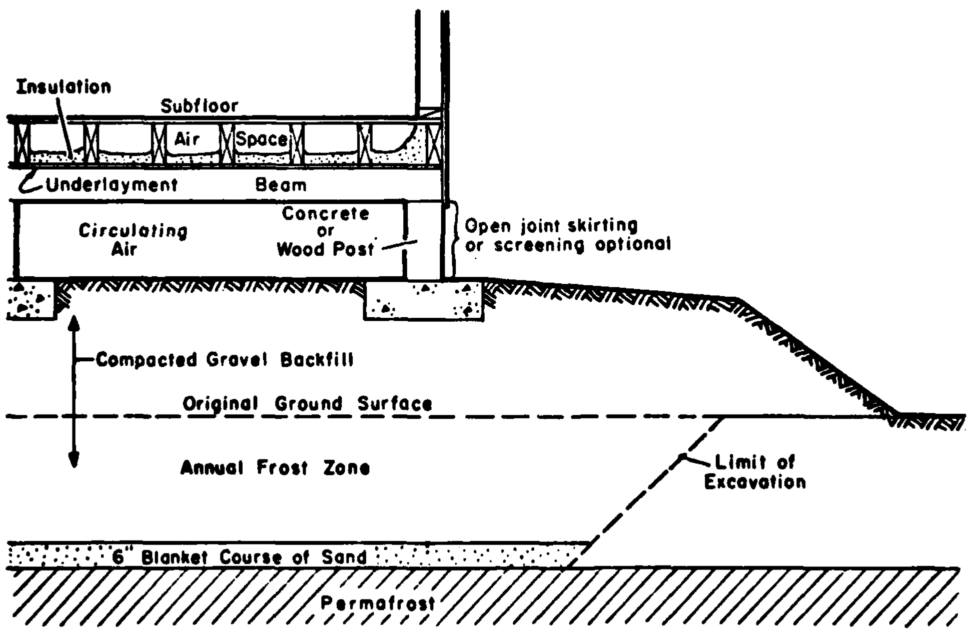


Figure 19. Typical design for light structure with air space and gravel mat.

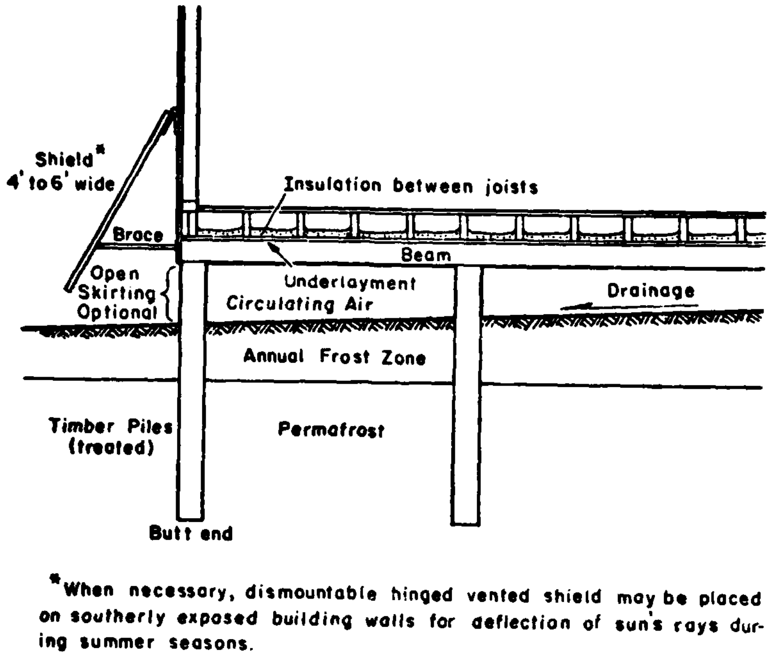


Figure 20. Typical ventilated foundation design for structures supported on piles.

Self-refrigerating or forced circulation thermal piles (para. 6) or freeze points may also be used for overall foundation cooling and control of permafrost degradation. Mechanical refrigeration should be used only in special circumstances because its success is dependent on the human operational factor and because it is costly to install and operate. However, refrigerants of various types circulated through copper tubing or freeze points installed in the ground and refrigerated air have been used in a number of cases to halt settlement caused by permafrost degradation.

The design of foundations for fuel and water tanks and refrigerated storage buildings may present a variety of special thermal problems. Some fuel tanks may be periodically filled with warm or hot fuel, with the heat dissipating slowly to the atmosphere and foundation over periods up to several weeks. Other tanks may be maintained continuously at warm, hot, or below freezing temperatures, potentially producing either thaw settlement or frost heave in the foundation, depending upon the conditions. The heating effect of sunlight may be significant. In seasonal frost and non-frost areas, facilities such as frozen food storage buildings placed on frost-susceptible soils can develop very damaging frost heave over a period of years unless adequate design precautions are taken; the situation is then the converse of the foundation thaw problem described above. When foundation conditions are unfavorable, a combination of foundation cooling and insulation can always provide needed control, in either a seasonal frost or a permafrost area.

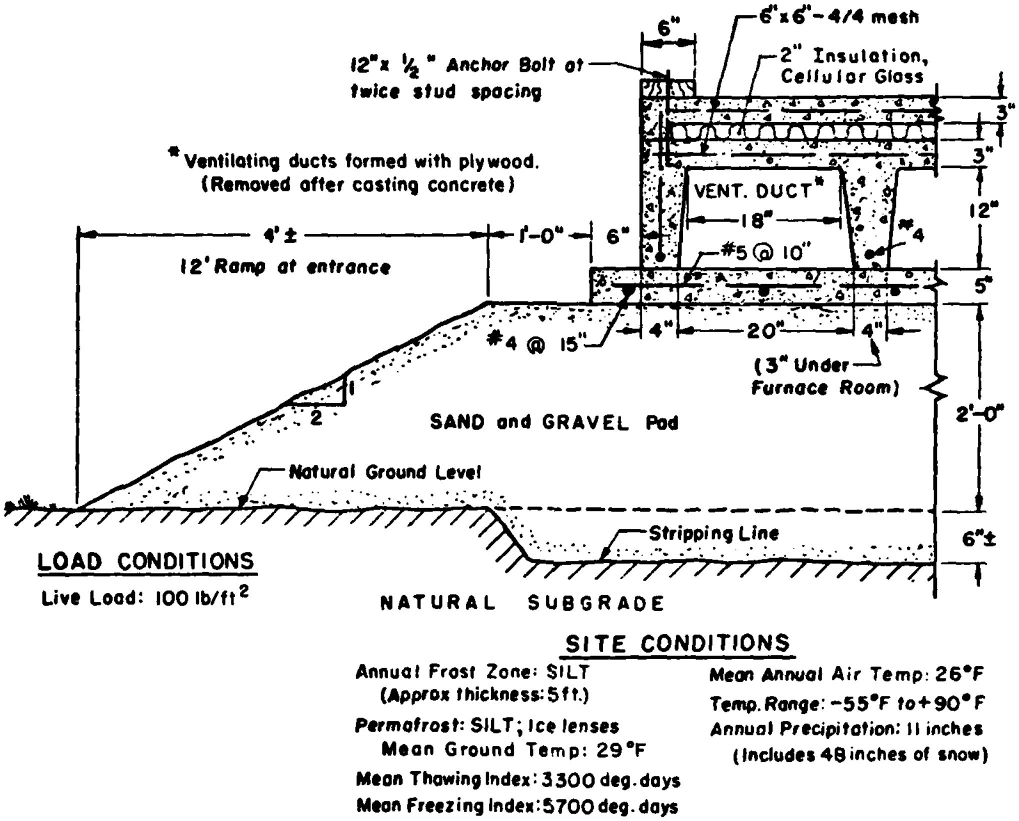


Figure 21. Ducted foundation for garage, Fairbanks, Alaska.

d. Foundation Insulation. Thermal insulation may be used in foundation construction in both seasonal frost and permafrost areas to control frost penetration and heave, for energy conservation and economy, for comfort, to control condensation, and to enhance the effectiveness of foundation ventilation. Loss of expected effectiveness because of moisture absorption must be avoided. Cellular glass should not be used where it will be subject to cyclic freeze-thaw in the presence of moisture (para. 9d). Insulation thicknesses and placement may be determined by the guidance given in TM 5-852-4 and -6/[8,10] and/or the latest edition of the ASHRAE guide[15].

e. Granular Mats. In areas of significant seasonal frost and permafrost a mat of non-frost-susceptible granular material may be used to moderate and control seasonal freeze and thaw effects in the foundation, to provide drainage under floor slabs, to provide stable foundation

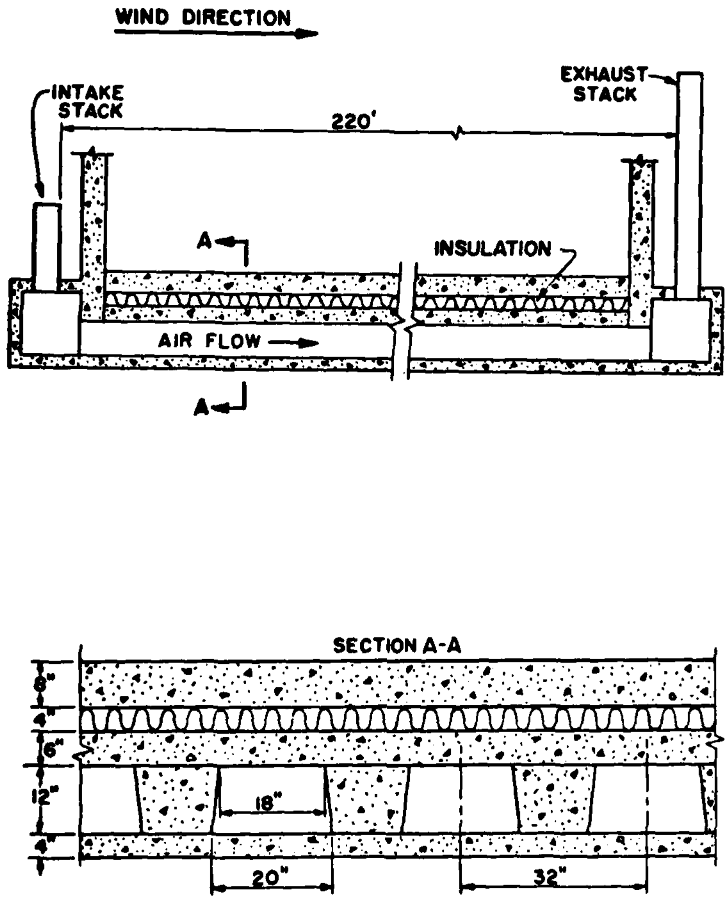


Figure 22. Schematic of ducted foundation.

support, and to provide a dry, stable working platform for construction equipment and personnel. Seasonal freezing and thawing effects may be totally or partially contained within the mat. When seasonal effects are only partially contained, the magnitude of seasonal frost heave is reduced through both the surcharge effect of the mat and the reduction of frost penetration into underlying frost-susceptible soils. Figures 19 and 21 illustrate typical granular mat usage in permafrost areas. TM 5-852-4[8] provides guidance in design of mats.

f. Solar Radiation Thermal Effects. Control of summer heat input from solar radiation is very important in foundation design in permafrost areas. In seasonal frost areas it may sometimes be feasible to take advantage of solar heat to reduce winter frost problems by coloring critical surfaces black.

In permafrost areas a very substantial part of the total heat input into the ground in summer may be contributed by solar radiation. When natural protective vegetative cover is removed or damaged by construction or replaced by earth fill or pavement, resultant increased summer flow of heat into the ground will increase the depth of thaw and may initiate degradation of permafrost. The right hand column of Table I indicates the differences in relative heat input which may be experienced. The edges of granular mats where the fill is tapered down to the natural ground surface are particularly susceptible, and settlement or sloughing of the embankment edges may result. When degradation in such cases extends below footings, the foundation and structure may be damaged.

Corrective measures which may be employed include shading (as illustrated in Figure 20, for example) and reflective surfaces. Establishment of live vegetative covering can be effective but may not be feasible in the time frame available or under the site climatic conditions. Piles which extend above the ground surface can absorb radiation and conduct heat down into permafrost but are easily painted a reflective color or provided with simple shielding. TM 5-852-4[8] provides more detailed guidance.

3. CONTROL OF MOVEMENT AND DISTORTION. The amount of movement and distortion which may be tolerated in the supported structure must be established and the foundation must be designed to meet these criteria. Movement and distortion of the foundation may arise from seasonal upward, downward and lateral displacements, from progressive settlement arising from degradation of permafrost or creep deflections under load, from horizontal seasonal shrinkage and expansion caused by temperature changes, and from creep, flow or slide of material on slopes. Heave may also occur on a nonseasonal basis if there is progressive freezing in the foundation, as under a refrigerated building or storage tank. If the soil conditions, moisture availability, frost penetration, imposed loading or other factors vary in the foundation area, the movements will be non-uniform. Effects on the foundation and structure may include various kinds of structural damage, jamming of doors and windows, shearing of utilities, and problems with installed equipment.

a. Frost Heave and Thaw Settlement Deformations. In the case of slab or footing foundations, frost heave may act directly upward against the base of the foundation as illustrated in Figure 13a or laterally against walls as in in Figure 13b. By a nominal estimate of the effective area of the slab of seasonally frozen soil which contributes to heave or thrust forces on a structure and by use of the maximum heave pressure data in Figure 12 a rough approximation of the total potential direct heave or thrust force on a given structure can be made.

In the type of situation illustrated in Figures 13a and 13b, detrimental frost heave effects can be avoided by use of a granular, non-frost-susceptible mat or backfill of sufficient thickness to avoid frost penetration into the frost-susceptible foundation materials or can be limited if full protection is not feasible. (see Walls and Retaining Structures in relation to Figure 13b.)

Piers, posts, piles or entire foundations may also be gripped laterally and heaved as in Figure 13c. The maximum uplift force which can be exerted in the latter cases is usually limited by the unit tangential adfreeze bond strength and the area of adfreeze contact on the member itself. Figure 23 illustrates tangential adfreeze bond stresses developed by frost heave acting on test timber and steel pipe piles installed in the annual frost zone and restrained against upward movement. The curve for the 1962-1963 test on an 8-in. (20 cm) steel pipe pile also clearly shows the relaxation in creep of heave stress following each successive wave of winter cooling.

For foundation members which experience full frost heave forces acting in tangential shear on lateral surfaces, including footings, walls, piles and anchors, the total frost force (without factor of safety) acting over the depth of the annual frost layer should be computed by assuming an average of 40 psi (275 kN/m^2) of tangential frost stress for

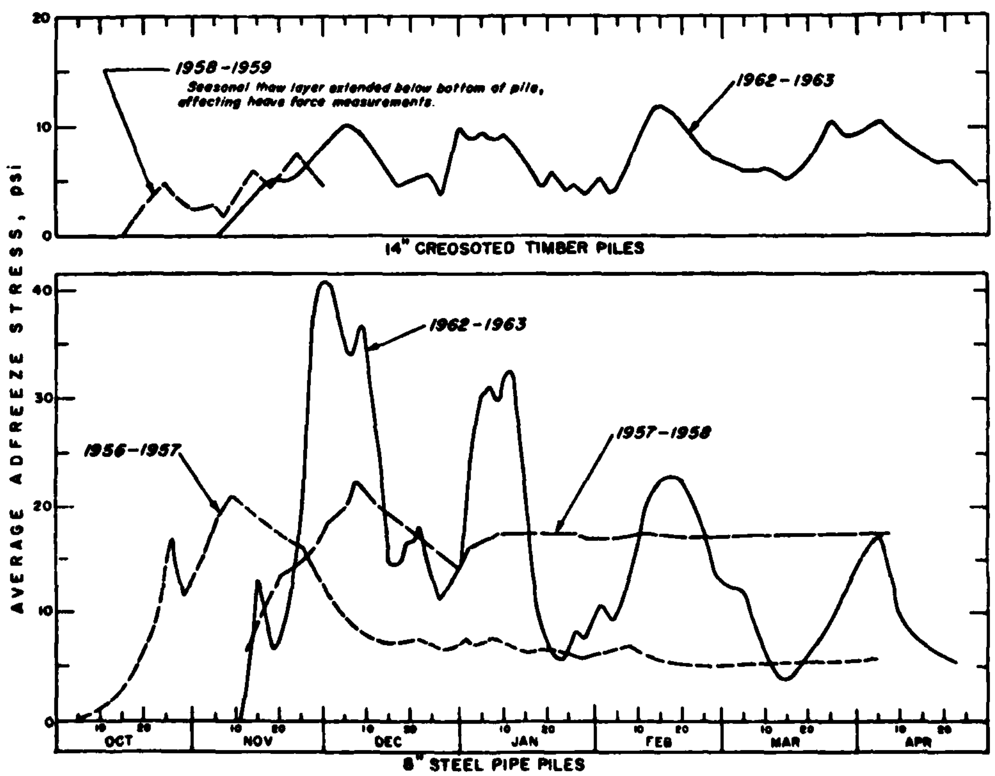


Figure 23. Heave force tests, average tangetial adfreeze bond stress vs. time, timber and steel pipe piles placed with silt-water slurry in dry-excavated holes. Piles were installed within annual frost zone only, over permafrost to depths from ground surface of 3.5 to 6.5 ft (1.07 to 1.98 m).(23)

steel in contact with silt and 60 psi (414 kN/m^2) for concrete in contact with silt. Values for other foundation members and soil materials should be in proportional conformance with the factors shown in Figure 29. The foundation members themselves should be designed on the assumption of 50% greater tangential frost stress acting on that part of the member which will be within the annual frost layer, to allow for deviations from the average stress.

Among methods which can be used to control detrimental frost action effects are: (1) placing non-frost-susceptible soils in the depth subject to freezing to avoid frost heave or thrust, (2) providing sufficient embedment or other anchorage to resist movement under the heaving forces, (3) providing sufficient loading on the foundation to counterbalance heaving forces, (4) isolating foundation members from uplift forces, (5) battering or tapering members within the annual frost zone to reduce effectiveness of adfreeze grip, (6) modifying soil frost susceptibility, or (7) in seasonal frost areas only, taking advantage of natural heat losses to minimize adfreeze and/or frost heave.

Various methods of isolation are available to reduce or eliminate the upward forces imposed on piling, footings and other foundations by frost heaving. They are most often used on piling, particularly when the depth of pile embedment in permafrost is insufficient to resist frost heaving. This may happen, for example, when bedrock is present at relatively shallow depth.

Two methods of isolation which have been used successfully in permafrost areas are shown in Figure 24. In both schemes the objective is to provide a low shear strength material next to the pile within the annual frost zone. The soil-oil-wax backfill provides slightly more

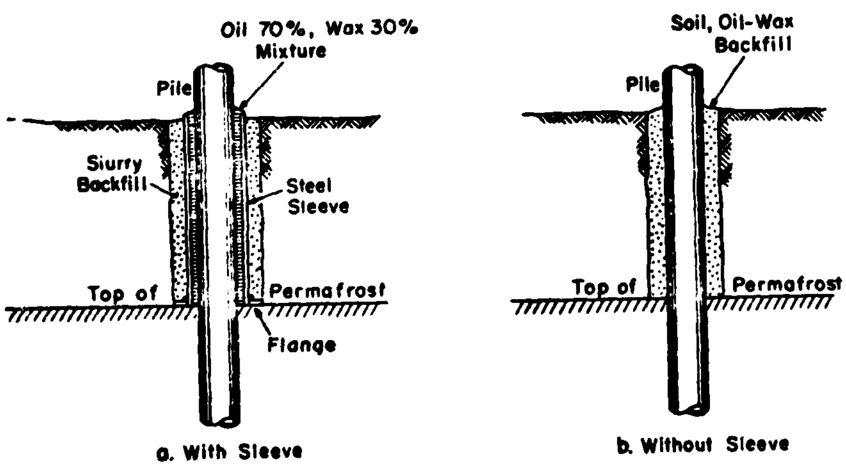


Figure 24. Heave isolation.

lateral support above permafrost than the oil-wax mixture, but can transmit more tangential shear. The increase of effective unsupported length cf pile which the oil-wax or soil-oil-wax backfill produces may sometimes be significant. Both backfills also serve to keep the space next to the pile from becoming filled with soil or ice. Both schemes can also be used in seasonal frost areas and can easily be used as remedial treatments if the foundation remains accessible. The flange on the casing in Figure 24a is for the purpose of minimizing the tendency of the casing to be gradually frost-heaved out of the ground.

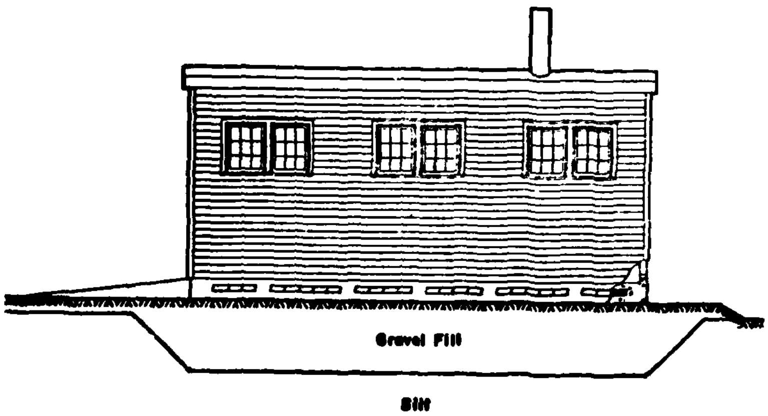
Because of their high capacity to transmit shear, relatively thin layers of untreated non-frost-susceptible backfill should not be expected to be effective in heave isolation.

Modification of the frost susceptibility of soils may sometimes be used to control heave. It has been shown that admixtures of Portland cement, asphalt or tar can render soils non-frost-heaving if the amount blended in is sufficient to reduce the water permeability to zero. The percent admixture required must be determined by test for each individual soil. The amount of admixture required represents substantial materials cost. Incorporation of the admixture into the soil is also a substantial expense and difficulty, recognized as constituting at least half the problem. Salts may be applied to foundation materials during construction to control freezing, but their effects are not permanent because of loss by leaching and diffusion. If a washing plant is available, a controlled amount of fines can be washed out of otherwise suitable soils, or hydraulic sluicing can be employed in the borrow pit to reduce fines in granular soils of marginal frost susceptibility.

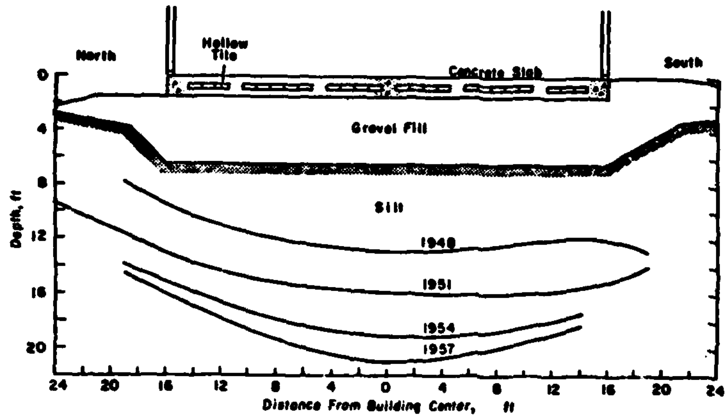
In permafrost areas, movement and distortion caused by thaw of permafrost can be extreme, as illustrated by Figure 25, which shows the performance of a wooden garage on a foundation containing hollow tile of inadequate size to provide effective foundation ventilation. Such movement and distortion should be avoided by designing for full and positive thermal stability. If damaging thaw settlement should start, a mechanical refrigeration system may have to be installed in the foundation or a program of continual jacking may have to be adopted. Discontinuance or reduction of building heating can also be effective. More detailed guidance is given in TM 5-852-4(8).

b. Creep Deformation. Only very small loads can be carried on the unconfined surface of ice-saturated frozen soil without progressive deformation. When the loaded area is buried to some depth, the allowable long term loading increases greatly but may be limited by unacceptable creep deformation well short of the allowable stress level determined from conventional short-term tests.

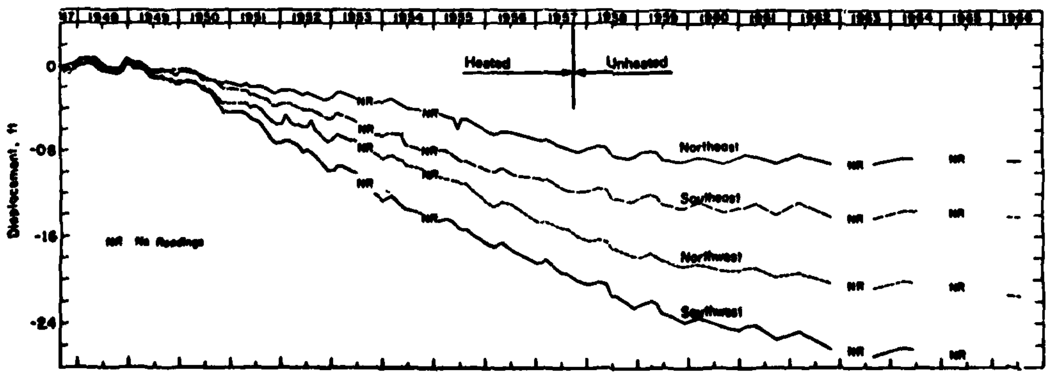
The ultimate strength and deformation characteristics of any particular saturated frozen soil depend primarily upon its temperature relative to 32°F (0°C) and the period of time that the soil will be subjected to a given stress. Ultimate strength increases and the rate of creep deformation decreases as the temperature decreases. Tests on frozen



a. East side elevation.



b. Degradation of permafrost on N-S centerline, 1948-1957.



c. Displacement of corners, 1947 through 1966.

Figure 25. Wood frame garage, 32 ft by 32 ft (9.75 x 9.75 m) on rigid concrete raft foundation, Fairbanks, Alaska.[31,37]

clays and silts indicate that the level of stress that can be resisted indefinitely can be as low as 5 to 10 percent of the failure strength measured under relatively rapid loading and its value is quite small near the thawing temperature. At intermediate stress levels creep ending in failure occurs. Even at stress levels sustainable in long term loading, however, unacceptable slow progressive creep deformation may occur. Present footing design practice is to (1) use large footings with low unit loadings, or (2) support footings on mats of well-drained non-frost-susceptible granular materials which reduce stresses on underlying frozen materials to conservatively low values, or (3) place foundations at sufficient depth in the ground so that creep is effectively minimized. Pile foundations are designed to not exceed sustainable adfreeze bond strengths. In all cases, analysis is based on permafrost temperature at the warmest time of the year. For cases which require estimation of foundation creep behavior, TM 5852-4[8] presents methods of analysis.

4. VIBRATION PROBLEMS AND SEISMIC EFFECTS. When the earth's surface is subject to seasonal freezing or is permanently frozen, the normal layered soil system is further complicated by the presence of frozen and/or thawing layers. Frequently these layers are not stable or constant but vary almost continuously as the temperature fluctuates during the year, and their presence will strongly affect the response of the entire mass to stress. Moreover the response of the soil to load varies tremendously from strengths approaching those of concrete or rock, to those of a slurry, depending upon temperature and soil properties.

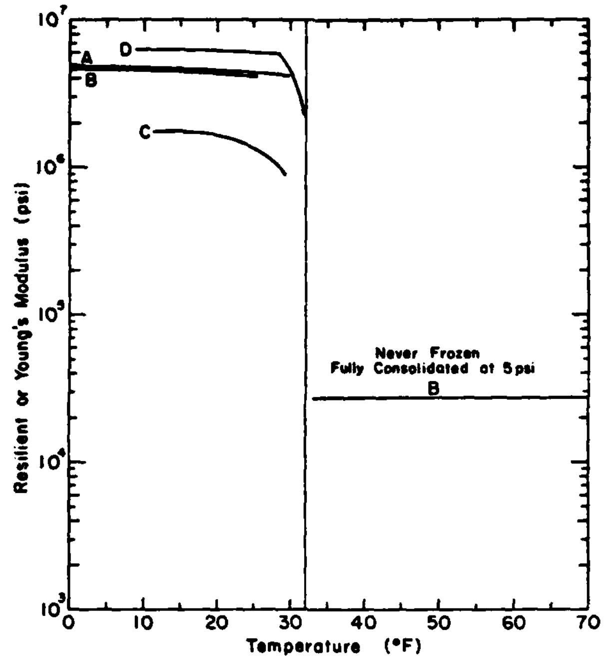
It is essential that the response to load of the soil under the various states be known, especially when the load is dynamic, involving propagation of stress waves in the soil mass. An important consideration in situations involving stress wave propagation through frozen ground is the abnormal condition of a very stiff layer overlying the relatively weak, nonfrozen layers. A difficult analytical problem is posed and phase velocity is likely to follow an opposite trend than would be expected where rigidity increases with depth[42]. Many areas in the cold regions are subject to seismic activity[2,47] and a substantial layer of frozen soil can greatly affect the surface response.

In general, design criteria for dynamic loads on nonfrozen soils and in temperate zone climates are applicable to seasonal frost or permafrost areas. Design criteria are given in TM 5-809-10[2] and EM 1110-345-310[3a] which also contain references to sources of data on the general behavior and properties of non-frozen soils under dynamic load and discuss types of laboratory and field tests available. Design criteria, test techniques and methods of analysis are not yet firmly established for engineering problems of dynamic loading of foundations. However, the properties of the frozen or thawing soils under such loads are greatly different. The stiffness of soil varies depending upon temperature, soil properties, and load conditions. The effect of above-freezing temperature change is small and usually neglected as insignificant but temperature at or below freezing becomes the most significant variable affecting the stiffness and strength. Below-freezing tempera-

ture causes the water in the soil to become ice, which has strong adhesive properties and cements the soil grains together into a much stronger or stiffer material. It follows that a completely dry soil does not significantly increase in stiffness at below freezing temperatures, but a normally wet soil increases in stiffness in an abrupt jump as the water freezes to ice. However, the ultimate stiffness is not reached immediately upon onset of the freezing temperature. All the water in the voids does not immediately freeze[16,45]. Accordingly, after the first abrupt increase, there is a more gradual increase as temperature decreases, the rate depending primarily upon the amount of unfrozen water in the soil voids. Depending upon soil type and saturation, the modulus may not reach a maximum until +20°F (-6.7°C) or even +15°F (-9.4°C). As temperature further decreases an increase in modulus may be observed but it is usually small. As a certain amount of time is involved in the flow of heat, the soil mass does not immediately freeze or thaw as the ambient temperature fluctuates about the freezing point. That is, the air temperature may rise above +32°F (0°C` for a considerable time without thawing a sizeable soil mass by a significant amount. Therefore, the effect of freezing and thawing on soils is not a simple relationship. Defining the soil as frozen or nonfrozen is not sufficient to establish the modulus or strength although it may establish the order of magnitude.

Figures 26, 27 and 28 show the effect of temperature on "Young's modulus" or the "resilient modulus" for sands, silts and clays subjected to a variety of dynamic loads under controlled laboratory conditions, from the rather sparse available data. The very large increase in modulus between the nonfrozen and the frozen state can be noted. The difference between the modulus of nonfrozen but fully consolidated soil and solidly frozen soil is about two to two and a half orders of magnitude for sands, silts and clays. However, the difference between the same nonfrozen soil and that at about +30°F (-1.1°C) is only about one to one and a half orders of magnitude depending upon the soil type. Likewise if frozen saturated fine-grained soil is thawed without full opportunity to drain and consolidate, the difference can be close to three orders of magnitude (see Figures 27, 28). Also, the difference in modulus between unfrozen and frozen soils can range from nearly zero for dry soil to the orders of magnitude described above, depending upon degree of saturation[44]. It can be further noted that the effect of temperature decrease below freezing varies depending on soil type. Sand (saturated) shows an abrupt increase in modulus between +33°F (0°C) and (-3.9°C) +25°F, whereas the modulus of clay increases more or less gradually as temperature decreases from +32°F (0°C) to 0°F (-17.8°C). The effect on silts is about halfway between that of sand and clay.

Analysis of the data shown in Figures 26, 27 and 28 indicates that the modulus of frozen soils, as for nonfrozen soils, varies with soil type, void ratio or density, and the properties of the load, i.e. stress σ, strain ε, and frequency. In general, the modulus increases with increasing grain size, decreasing plasticity, decreasing void ratio (or increasing density), decreasing stress and strain, and increasing fre-



Frozen

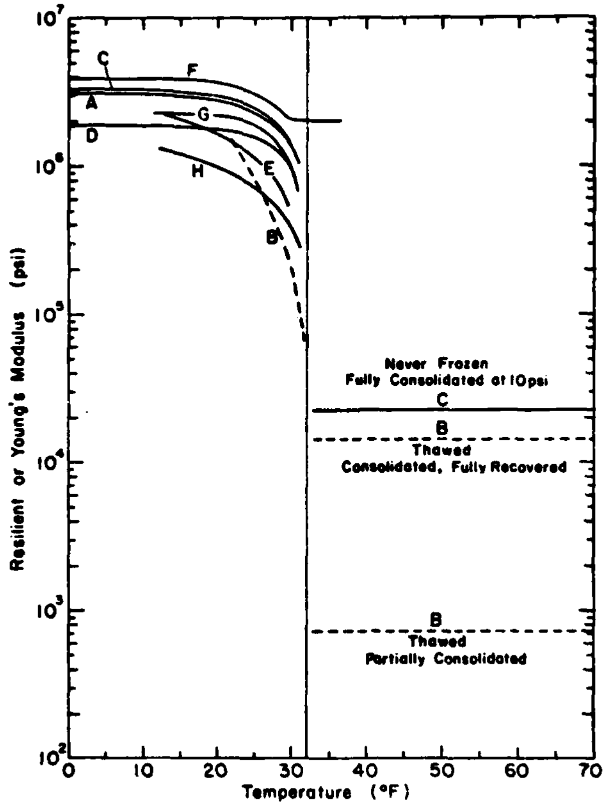
Investigator	Curve design.	Soil: Name	Soil: e	Soil: S, %	Soil: PI	Load: σ_0, psi	Load: $\sigma_1-\sigma_3$, psi	Load: ϵ, %	Load: Frequency, Hz
Kaplar (27)	A	MacNamara	0.47	86.0	NP	0			5 k
Stevens (42)	B	20-30 Ottawa	0.53	99.7	NP	0	5.0	varies	1 k
Vinson (46)	C	20-30 Ottawa	0.49	100.0	NP	50	varies	1×10^{-2}	0.3
Nakano and Froula (33)	D	20-30 Ottawa	0.50	100.0	NP	0			1 m

Nonfrozen

Investigator	Curve design.	Soil: Name	Soil: e	Soil: S, %	Soil: PI	Load: σ_0, psi	Load: $\sigma_1-\sigma_3$, psi	Load: ϵ, %	Load: Frequency, Hz
Stevens (42)	B	20-30 Ottawa	0.60	93.8	NP	5	1.0	varies	1 k

Figure 26. The effect of freezing temperatures on the stiffness of sands.

quency. Confining pressure ($\sigma_0 = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$) also has an effect but, because of the relatively high modulus and strength of frozen soils, this effect is small in the normal range of pressures encountered. Test data are insufficient to establish reliable relationships for these variables but some indication can be obtained from the data in Figures 26, 27 and 28 by replotting results for the variable desired. Note that the effect of these variables is small compared to the effect of temperature fluctuation close to the freezing point.



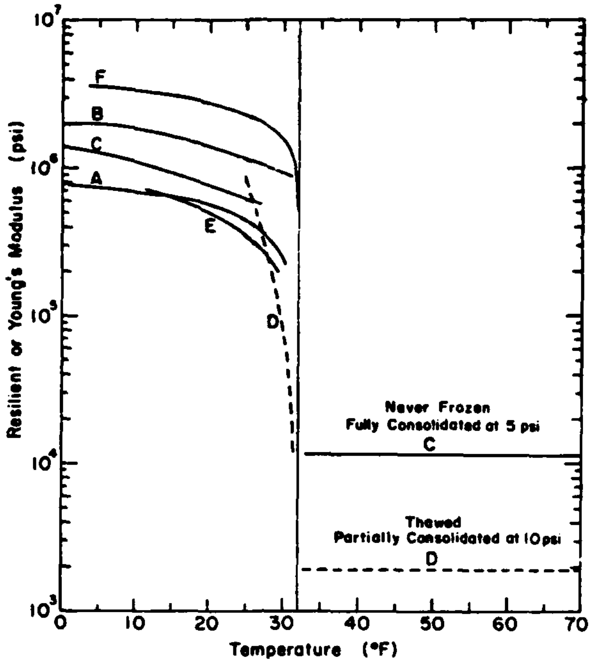
Frozen

Investigator	Curve design.	Soil: Name	e	S, %	PI	Load: σ_0, psi	$\sigma_1-\sigma_3$, psi	ϵ, %	Frequency, Hz
Kaplar (27)	A	Fairbanks silt	0.65	99.0	NP	0			5 k
Johnson, et al. (25)	B	Hanover silt	1.20	95.0	5.0	10	14.5	varies	0.3
Stevens (42)	C	Hanover silt	0.71	96.6	NP	0	5.0	varies	1 k
Stevens (42)	D	Fairbanks silt	1.00	99.0	NP	0	5.0	varies	1 k
Vinson (46)	E	Hanover silt	0.60	100.0	NP	50	varies	1×10^{-2}	0.3
Nakano and Froula (33)	F	Hanover silt	1.06	100.0	NP	0			1 m
Czajkowski (21)	G	Hanover silt	0.60	100.0	NP		varies	3.16×10^{-3}	0.3
Czajkowski (21)	H	Hanover silt	0.60	100.0	NP		varies	3.16×10^{-2}	0.3

Nonfrozen

Investigator	Curve design.	Soil: Name	e	S, %	PI	Load: σ_0, psi	$\sigma_1-\sigma_3$, psi	ϵ, %	Frequency, Hz
Johnson, et al. (25)	B	Hanover silt	1.20	95.0	5.0	10	14.5	varies	0.3
Stevens (42)	C	Manchester silt	0.73	90.4	NP	10	1.0	varies	1 k

Figure 27. The effect of freezing temperatures on the stiffness of silts.



Frozen

Investigator	Curve design.	Soil: Name	Soil: e	Soil: S, %	Soil: PI	Load: σ_0, psi	Load: $\sigma_1-\sigma_3$, psi	Load: ϵ, %	Load: Frequency, Hz
Kaplar (27)	A	Fargo	1.127	92.0	46.0	0			5.5 k
Kaplar (27)	B	Boston Blue	1.87	96.0	27.0	0			4.5 k
Stevens (42)	C	Goodrich	0.84	98.4	18.0	0	5.0	varies	1 k
Chamberlain Cole, Johnson (20)	D	Morin	0.736	86.0	16.9	10	19.5	varies	0.3
Vinson (46)	E	Ortonagen	1.60	100.0	37.0	50.0	varies	1×10^{-2}	0.3
Nakano (33)	F	Goodrich	0.84	100.0	18.0	0			1 m

Nonfrozen

Investigator	Curve design.	Soil: Name	Soil: e	Soil: S, %	Soil: PI	Load: σ_0, psi	Load: $\sigma_1-\sigma_3$, psi	Load: ϵ, %	Load: Frequency, Hz
Stevens (42)	C	Goodrich	0.99	98.2	18.0	5	1.0	varies	1 k
Chamberlain Cole, Johnson (20)	D	Morin	0.736	81.0	16.9	10	8.9	varies	0.3

Figure 28. The effect of freezing temperatures on the stiffness of clays.

The velocity of propagation of stress waves in frozen soils varies from that in nonfrozen soil in the same way as the modulus. Seismic surveying on frozen ground differs from normal only in that equipment and methods must be suitable for the much higher velocities involved as well as operational in cold temperatures(41). However, a decrease in velocity with depth violates the basic condition for application of the seismic refraction method and may prevent its use. When the void ratio of saturated frozen soil is greater than 1.0 and the volume of ice

exceeds the volume of soil, the stress wave tends to travel in the ice matrix rather than the soil structure. Accordingly the velocity tends to approach that of ice of a similar structure and it is difficult to discern a velocity change between ice-rich frozen soil and an ice lens. In general, the velocity of propagation, and/or the modulus of saturated frozen soil, is greater than that of ice. However, in nature, ice in frozen soils is often so segregated as to complicate the passage of stress waves, and velocities can vary accordingly.

An important parameter of a material subjected to dynamic loads is its damping capacity. Only a few measurements of this capacity have been made for frozen soils, but data available indicate that the damping capacity of frozen soil is about equal to that of the corresponding nonfrozen soil, although the modulus may differ by several orders of magnitude[44]. Otherwise the damping parameter varies as for nonfrozen soils; that is, it increases with increasing strain and decreasing strain rate or frequency and is less for cohesionless soils than for cohesive soils[44, 48]. The combination of high modulus and relatively high energy absorption characteristics makes frozen soil a unique material whose properties must receive special consideration in foundation design in cold regions.

Very few determinations of Poisson's ratio for frozen soils have been made to date (1977) and such data as are available[20, 44] indicate that they do not differ greatly from those commonly assumed for non-frozen soil.

Prediction of the response of soils in cold regions to dynamic loads is extremely difficult and complicated. Depending upon the sensitivity of the foundation design to values from such predictions, it may be necessary to conduct special tests or engage consultants with specialized knowledge of the subject. It is essential that the designer consider that a given normal soil can vary from a material having a shear modulus of 2 million psi to a material having a shear modulus approaching zero during one freeze-thaw cycle.

5. DESIGN OF FOOTINGS, RAFTS AND PIERS. If foundation soils are clean, granular, non-frost-susceptible (and, in permafrost areas, free of excess ice) conventional temperate zone foundation practice may be used in both seasonal frost and permafrost areas. For foundations for permanent buildings which would be adversely affected by anticipated temperature-and moisture-induced seasonal expansion, contraction and volume changes, which are most intense in the upper layers of the ground, foundations should be placed at a minimum depth of 4 ft (1.2 m) in the non-frost-susceptible soils. For foundations and facilities more tolerant of small seasonal movements, foundations may be placed at shallower depths or even at the surface of the non-frost-susceptible materials.

For frost-susceptible soils in seasonal frost areas, footings and piers should normally be supported below the design depth of seasonal frost penetration, whether the structure is heated or not. Such footing

depth may appear unnecessary for many kinds of heated structures in seasonal frost areas because of the protective effect of heat losses into the ground, but it is not realistic to assume that a facility will continue to be heated, or fully heated, if it is placed on stand-by status at some future time. Energy convervation may also require insulation measures to preclude such heat losses. Placement of the structure on a pad of non-frost-susceptible fill of sufficient thickness can, of course, avoid this footing depth requirement. A non-frost-susceptible mat is also appropriate under a raft or mat foundation.

Fine-grained soils, even though they may have substantial frost-heave potential, are preferable for backfill on exterior footing walls and foundations because they minimize the depth of frost penetration (as well as helping to keep basements dry when used around buildings).

Footing, pier and raft foundations can be used successfully in permafrost areas with proper design care, although they may not always be as economical as pile foundations. Foundations may be supported at the surface of granular frost-susceptible mats as in Figures 19 and 21, may be placed within the mat, or may be placed at or below the depth of maximum seasonal thaw which will exist after construction. In the latter case, a layer of granular non-frost-susceptible material should be used immediately below the footing. When appropriate, analysis should be made of safety against unacceptable creep deformation, in accordance with guidance in TM 5-852-4[8].

Footing or pier members in the annual frost zone which may be placed in tension by frost heave forces should be made strong enough to withstand such forces, but piers should also be of minimum diameter to reduce the area of potential frost adhesion. Concrete members should contain sufficient steel to prevent tension cracking and exposure of the steel to moisture. Bases of footings should be sufficiently large to resist frost heave uplift through passive soil reaction against the base projections, and connections to vertical members should contain sufficient steel to resist the tension forces during heave. Pier-type foundations should extend to sufficient depth below the annual frost zone to resist such uplift through skin friction or adfreeze bond on lateral surfaces. Uplift resistance contributed by dead load from structure and weight of foundation should be taken into account. Where practical, surfaces in the annual frost zone may be battered to help minimize heaving forces. Frost heave forces may be evaluated as described in paragraph 3a; isolation of foundation members from the annual frost zone by the methods described therein may sometimes be practical.

Guidance with respect to allowable bearing values, factors of safety, settlement and creep analysis and other aspects of footing, pier and raft design, including illustrative design examples, is given in para. 4-7 of TM 5-852-4[8]. Guidance concerning factors of safety and methods of bearing capacity analysis given in TM 5-818-1[3] is also applicable.

6. PILING. Pile foundations offer many advantages where construction is on frost-susceptible soils in areas of deep seasonal frost penetration or permafrost with high ice content. Piling permits the structure load to be transferred to depths where soil supporting strength remains relatively stable seasonally and through the life of the structure.

With regard to materials, the most commonly used types are timber piles and steel H, I, or pipe piles. Concrete piles should not be used under conditions where frost heave forces may produce tensile stresses sufficient to crack the piles and expose the reinforcing steel to corrosion; it is possible for the upward forces to be double the downward-acting design loadings. Pressure-treated timber piles are usually readily available, have relatively low thermal conductivity, are easily installed, and when installed butt-down provide some natural restraint against frost heaving, a medium to heavy coating of creosote does reduce bearing capacity. Steel piles offer an advantage in their capacity to be installed in frozen ground under favorable conditions by driving.

Thermal piles offer special advantages in heat transfer capability, in addition to load bearing capacity. Sometimes only the heat transfer function may be needed. Their customary functions are to assist freeze-back at time of installation in permafrost and to increase thermal stability under service conditions by speeding and intensifying the winter cooling of the foundation. Thermal piles may be either self-refrigerated, self-initiating systems which automatically cease operation when air temperatures become warmer than those around the lower part of the pile, or forced fluid circulation systems. The self-refrigerated systems are of two types: single phase and two-phase. The single phase system operates as a liquid convection cell and the two-phase system operates on an evaporation-condensation cycle.

For installation of piles in permafrost, the most satisfactory techniques are (1) to place the pile in a dry-augered hole, place a soil-water slurry around it and allow it to freeze, or (2) to drive the pile (steel only) with heavy driving equipment, if the soil is fine-grained and not colder than about 25°F (-3.9°C). Other methods of making holes for slurried piles can also be employed. However, steam or water thawing should not be used, except possibly in very cold permafrost, because the large amounts of heat introduced into the permafrost may make freeze-back in a reasonable time impossible. Without solid freeze-back, piles will neither develop full load capacity nor resist frost heave. When piles are installed by driving, the amount of heat introduced into the permafrost is usually negligibly small and freeze-back is complete a short time after completion of driving. If the permafrost temperature is close to 32°F (0°C), natural freeze-back of slurried piles is slow and may have to be limited to the spring when permafrost temperatures are coldest, unless mechanical refrigeration is used. Mechanical refrigeration may also be required if spacing of piles is too close. Mechanical refrigeration is usually provided by installing pipes or tubing on the piles, through which refrigerant is circulated after pile emplacement.

To resist frost heave, piles should be embedded in permafrost at least 10 ft (3.05 m). They must also have sufficient embedment to support the applied loads without excessive cumulative creep in adfreeze bond, over the life of the structure. Frost heave forces may be estimated as described in para. 3a. Figure 29 shows average long-term sustainable and ultimate tangential adfreeze bond strengths vs temperature as measured on piles installed in silt permafrost with silt-water slurry backfill.

Pile load tests may be required or desirable to obtain data needed for design, to verify design assumptions, and/or evaluate various alternative designs. Because of the special creep behavior of frozen soils, special pile test procedures must be followed.

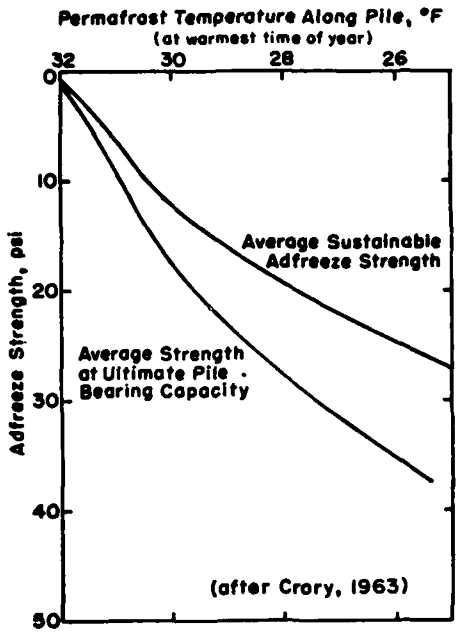
TM 5-852-4[8] presents detailed guidance on emplacement methods for piles in permafrost, freeze-back, design depth of embedment, load testing, factors of safety and thermal piles.

7. WALLS AND RETAINING STRUCTURES. In both seasonal frost and permafrost areas, bridge abutments, retaining walls, bulkheads and similar structures are subject to overturning frost forces if the soils are frost-susceptible, in addition to heave and settlement, as indicated in Figure 13b. The more or less horizontal thrust force can be sufficient to tilt, move or crack the wall. Progressive annual tilt movements can accumulate to very substantial amounts.

The most satisfactory method of handling this problem is to place non-frost-susceptible backfill directly behind the wall as shown in Figures 30a and b, to a thickness equal to the depth of frost penetration, using a filter layer or filter cloth next to fine-grained soil when needed. Design thickness should take into account the greater frost penetration that will occur in non-frost-susceptible soil. Positive drainage of the backfill should be provided, but the possibility that the drainage system may be blocked by freezing during a significant part of the year must be taken into account in wall stability analyses. Guidance in wall stability analysis procedures is given in TM 5-818-1[3].

The chart in Figure 31 may be used to estimate the thickness of non-frost-susceptible backfill required behind concrete walls in order to confine seasonal freezing to the backfill. The wall surface freezing index must first be computed using an appropriate n-factor. If the wall receives no sunshine during the freezing period, is exposed to substantial wind, and remains free of snow and ice (assuming an essentially vertical wall face), an n-factor of 1.0 should be used. For average conditions, with average exposure to the sun, an n-factor of 0.9 may be assumed. If the wall has a southerly exposure and thus receives much sunshine, the n-factor may be as low as 0.5 to 0.7 in southerly latitudes or 0.7 to 0.9 in very high latitudes.

Figure 31 may also be used for estimating the depth of frost penetration vertically into drained granular soil below a snow-free horizontal surface, using an appropriate n-factor. For bare ground the chart is entered with zero wall thickness.

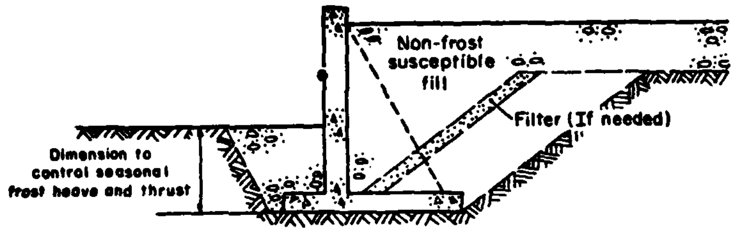


Correction Factors for Type of Pile and Slurry Backfill
(using steel in slurry of low-organic silt as 1.0)

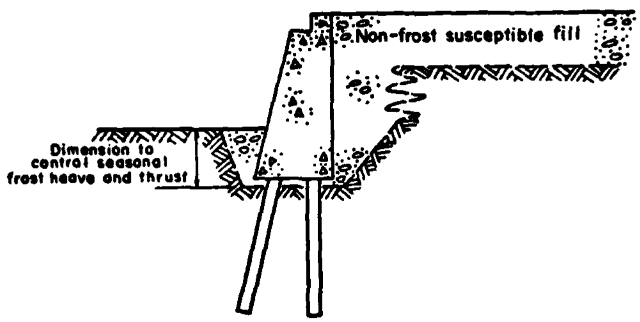
Type of pile	Slurry soil Silt	Slurry soil Sand
Steel	1.0	1.5
Concrete	1.5	1.5
Wood, untreated or lightly creosoted	1.5	1.5
Wood, medium creosoted (no surface film)	1.0	1.5
Wood, coal-tar treated (heavily coated)	0.8	0.8

NOTES:
1. Applies only for soil temperatures down to about 25°F.
2. Where factor is the same for silt and sand, the surface coating on the pile controls, regardless of type of slurry. In the remaining factors the pile is capable of generating sufficient bond so that the slurry material controls.
3. Pile load tests performed using 10-kip/day load increment were adjusted to 10-kip/3 day increment to obtain curves shown.
4. Clays and highly organic soils should be expected to have lower adfreeze bond strengths.

Figure 29. Tangential adfreeze bond strength vs. temperature for silt-water slurried 8.625 in. (21.9 cm) O.D. steel pipe piles in permafrost averaged over 18 to 21 ft (5.5 to 6.3 m) embedded lengths in permafrost.



a. Counterfort wall



b. Pile-supported wall

NOTE: Provide positive drainage of non-frost-susceptible fill behind walls.

Figure 30. Wall treatments.

Where direction of freezing is vertical rather than horizontal behind a wall, some relief of frost uplift may be obtained by battering the inner face. However, it should not be assumed that this will eliminate frost uplift forces. Maximum potential uplift forces may be estimated as described in para. 3a.

8. FOUNDATIONS AFFECTED BY WATER BODIES. When piles, caissons or piers are located in water bodies subject to fluctuation in level, as in rivers or tidal areas, the adherence of ice to the foundations during periods of low water may cause serious uplift forces because of buoyancy of the ice when a rise in water level occurs. Damage to structures may be severe and may occur at any latitude where significant freezing occurs. Piles installed without the added weight of the superstructure can heave or be jacked completely out of the ground in one winter. Where such uplift is possible, piles, piers, or caissons should be kept entirely smooth at the potential ice adherence levels and must have adequate depth of embedment or other anchorage to resist the uplift. Cross ties or bracing should be designed to allow for differential heave between piles and to withstand stresses imposed by massive ice accumulations.

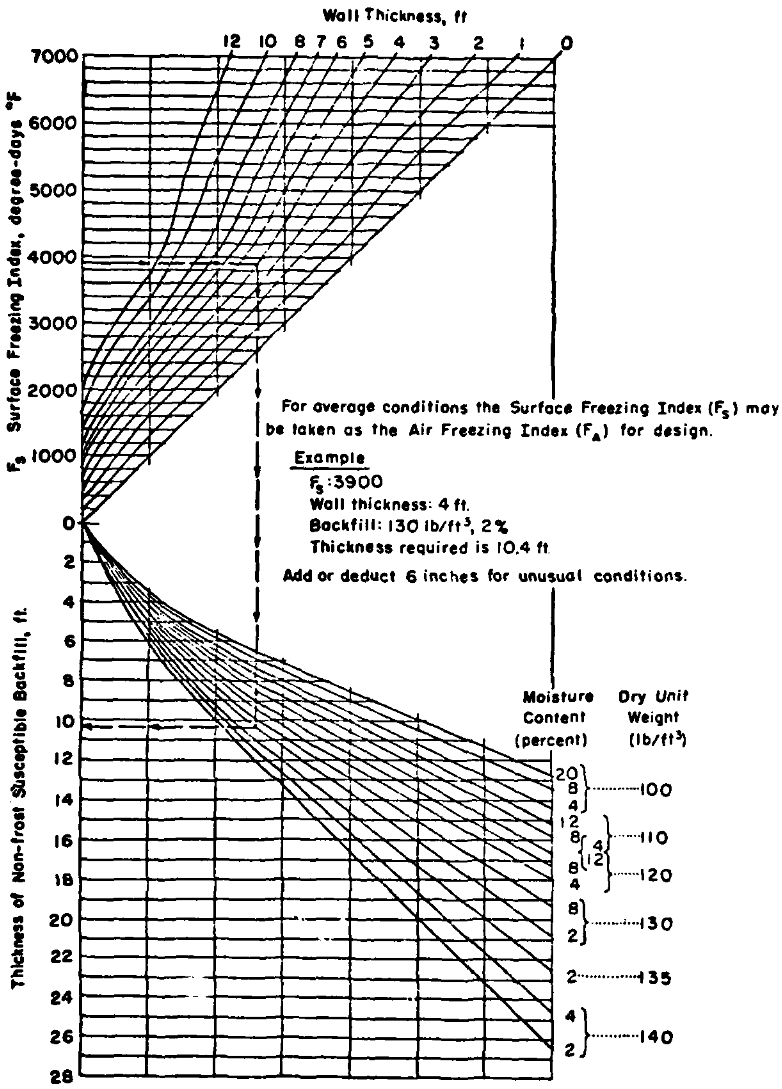


Figure 31. Thickness of non-frost-susceptible fill behind concrete walls.

Floating ice floes and debris during spring ice break-up or other times of flood waters can seriously damage bridge or other structures having foundations in water bodies, by impact or jamming and rafting against the structure. Design should include extra strength members, streamlined shapes and/or protective structures.

In permafrost areas, foundations in or near water bodies are affected by the special permafrost conditions commonly found there. The perma-

frost table tends to be depressed under a water body. Temperatures of permafrost are likely to be warmer near and under the water, and permafrost may even be absent. Especially in marginal permafrost areas, there may be little or no thermal capacity for natural freeze-back of piles, and the tangential adfreeze strength that can be relied on may be very low. Slight changes in the subsurface thermal regime may cause major changes in foundation stability. For these reasons, especially careful subsurface exploration is required at proposed locations of foundations for bridges, docks or other structures in or near water bodies. Even with extremely detailed subsurface information, however, achievement of permanently stable foundations may be a difficult engineering challenge.

The problem of thermal stability of culvert foundations in permafrost areas is somewhat similar to that of bridges and waterfront structures in that the presence and flow of water causes a special thermal regime under the structure. If the culvert is constructed in a natural drainageway, a special, relatively stable thermal pattern may already exist, and construction of the culvert may involve minimum change. Therefore, a natural drainage location is nearly always preferable for a culvert, even though the ground temperatures may be warmer. If built on ice-rich soil without precautions, the culvert may be rapidly destroyed by differential settlement and piping during the thaw season. Use of pile-supported bridges in lieu of culverts, or excavation and replacement of thaw-susceptible foundation materials, may sometimes be justified in difficult situations. The design of culverts in arctic and subarctic areas is discussed in greater detail in TM 5-852-4[8], and TM 5-852-7[11].

9. TOWERS, MASTS AND UTILITY POLES. Towers and masts for transmission lines, communication antennas, cableways or other purposes are commonly either self-supporting or guyed. Depending on the design and purpose of the tower or mast, seasonal vertical movement of the foundation may or may not be detrimental. If heave of the foundation is differential, problems of tipping, structural overstressing, or misorientation may arise. If the tower or mast is guyed, some guys and anchors may be overstressed; other guys will be slack. To control these effects, granular non-frost-susceptible mats may be used for the tower or mast foundation, either on top of the natural ground or set into the ground. Pros and cons of various footing and pad schemes are outlined in TM 5-852-4[8]; suggested schemes for pile and timber crib type foundations are also included.

Pole lines in both seasonal frost and permafrost areas are very susceptible to frost heave and progressive jacking out of the ground. When lateral support of thaw-softened ground becomes inadequate, the pole will fall, often taking down the line. To control this, a number of alternatives may be considered:

a. Providing sufficiently firm pole embedment below seasonal frost to resist heave (as by sufficient depth of embedment or by grouting into rock).

b. Using isolation techniques in the annual frost zone.

c. Supporting the poles above the natural ground surface by means of cross-braced, rock-filled timber cribs or non-frost-susceptible granular pads.

d. Use of tripod of 3 poles.

Guying of poles can help to avoid their sudden collapse but introduces the additional problems of stability of the anchors themselves and of varying guy-wire tension.

10. ANCHORAGES. While anchors installed in frozen soil may be capable of sustaining relatively high short-duration loads without difficulty, they can exhibit unacceptable yield and creep under much lower long-term loadings. Therefore, anchors installed in permanently frozen ground require special care in design. Only anchors which develop very low level stresses over a relatively large area should be used. Conventional metal expanding anchors should not be used in frozen soil or rock containing ice as the extremely high local stresses developed with such anchors cause rapid plastic deformation and creep in the ice component. For permanent anchors in frozen ground, design should be predicated on whichever is controlling: ultimate strength or holding creep within acceptable limits. Design guidance for conventional and special frozen ground anchors for installation in permafrost, including factors of safety, is given in TM 5-852-4[8].

In seasonal frost areas, anchors placed in frost-susceptible soils should be installed below the annual frost zone. Unless isolated from the annual frost zone by treated backfill or similar measures, the anchor rod itself should be designed for uplift and the total frost uplift force should be computed in accordance with para. 3a. The total frost uplift force should be added to the design tensile load imposed on the anchor.

11. GRADE BEAMS. Horizontal structural members placed at or just below ground level and which may be subject to uplift should be avoided when frost-susceptible soils are involved.

12. FOUNDATIONS FOR NON-HEATED FACILITIES. Foundations for non-heated facilities may involve special design approaches.

a. Non-Heated Buildings. Temperatures at ground floor level in an unheated building will depend on such factors as roof and wall insulation, degree of ventilation, exterior reflectivity, window space, and seasonal percent sunshine. For an unheated building fully open to the outside air, without excessive window area exposed to sunlight, and in which the winter temperatures at floor level may be assumed the same as shaded meteorological station air temperatures, a mat of non-frost-susceptible material thick enough to contain seasonal freeze and/or thaw may be used over frost-susceptible soil as shown in Figure 32, with an uninsulated slab-on-grade floor.

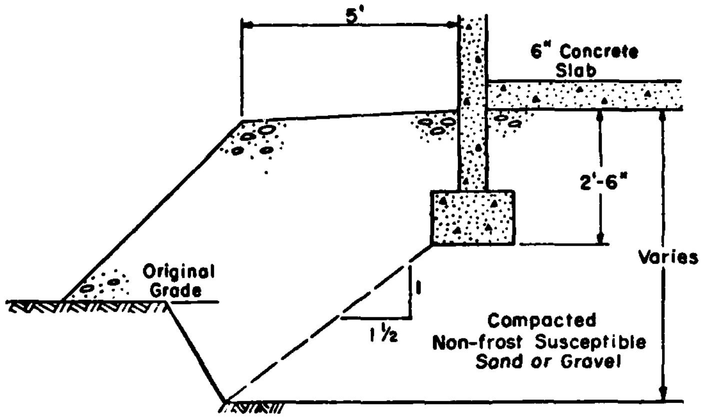


Figure 32. Typical foundation design for unheated building over frost-susceptible soil in deep seasonal frost or permafrost areas.

This design can be modified to conditions in any frost area or to slightly warmer, less well ventilated buildings by altering the mat thickness. Other possible design alternatives or modifications for permanent or temporary non-heated facilities are as follows:

(1) Addition of under-floor insulation.

(2) Use of a structural floor supported above the ground on footings or piles so that it can be isolated from frost heave. This system can also be used to provide a ventilated foundation.

(3) Use of a gravel floor of nominal thickness directly on the natural soil, accepting resultant frost heave.

(4) Use of a reinforced concrete floor slab with sufficient non-frost-susceptible material to reduce total and differential heave to tolerable amounts.

b. <u>Exterior Footings and Piles</u>. Footings and piles placed outside of buildings for support of porches, roof extensions and unheated connecting corridors receive none of the heating benefits received by the main foundation and may experience maximum frost heave because snow is kept removed in winter.

Foundations of this type should be designed on assumption of full 30-year depth of frost penetration with conservative n-factors (para. 2a). If only small pipe columns are involved they may be isolated from the annual frost zone as described in paragraph 3a.

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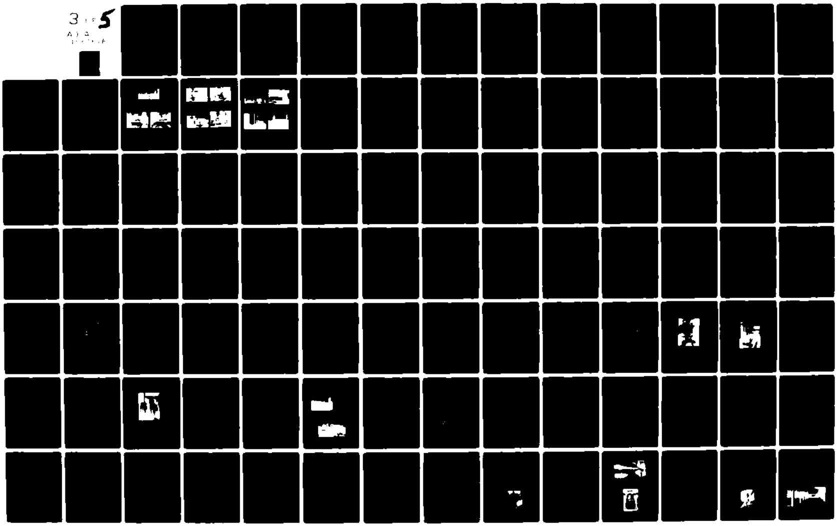

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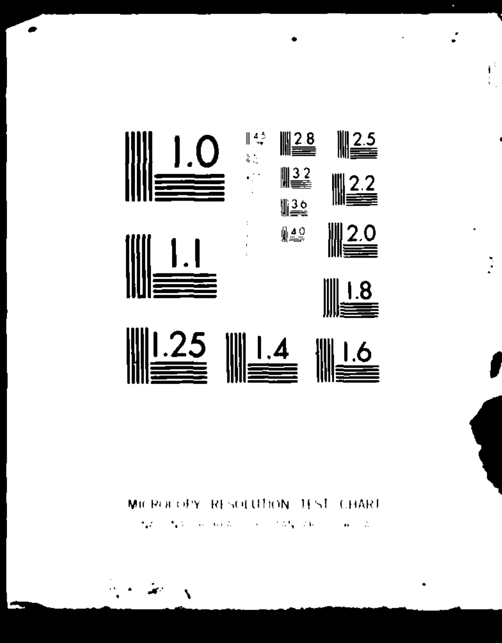

c. Exterior Aprons. Difficulty frequently arises when an exterior unheated apron or transition pavement is connected to a structure which is heated or is otherwise protected against frost heave. As indicated in Figure 33, heave may cause an abrupt displacement of the apron at the junction with the building and may block outward-opening doors. It may also cause structural damage and apron cracking and may interfere with drainage and thus cause icing. Possible solutions include:

(1) Use of a pad of drained non-frost-susceptible material placed under the apron to the full depth of frost penetration.

(2) Construction of part or all of the apron as a structural slab supported with sufficient space under it so that the heaving soil will not come in contact with it.

(3) Provision of a hinged ramp or a step at the connection with the building, the level of which will vary seasonally.

(4) Cantilever the apron from the main structure.

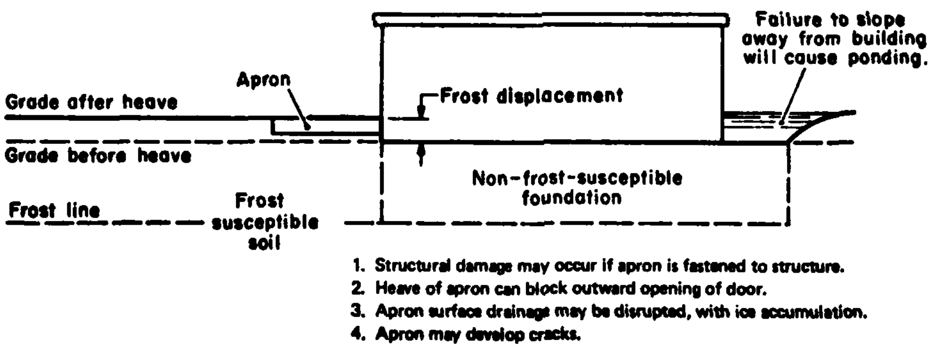

Figure 33. Exterior apron problems.

d. Exterior Platform and Steps. These can be supported on footings or piles as in b above, but alternative (3) under c above or cantilevering the platform or steps from the building may be equally satisfactory.

13. UTILITY LINE AND PIPELINE FOUNDATIONS. A rigid, insulated and heated conduit inclosing service utilities such as water, steam, sewer and fire protection lines is commonly called a utilidor. Utilidors and individual pipelines may be placed either above or below ground. Above ground placement is common in permafrost areas because seasonal frost effects and transmittal of heat into thaw-susceptible foundation materials can be more easily controlled, because access is easier for repair and

maintenance, and because underground moisture and ground thermal cracking problems are avoided. However, exposure to environmental extremes is greater, and the lines create surface obstructions. Above ground placement can also be advantageous in areas of deep seasonal frost penetration. Below ground placement is advantageous in areas of shallower seasonal frost penetration where lines can be placed below frost depth, and in permafrost areas where the soils are well-drained, non-frost-susceptible sands and gravels. In the latter areas only nominal depth of burial is used.

To control heat input from a hot pipeline or utilidor into permafrost a combination of insulation and elevation of the line above the permafrost on gravel berms or piles may be used, possibly including thermal piles and/or reflective surfaces on the berms. Insulation and elevation may also be employed in non-permafrost areas to avoid causing frost heave when fluid or gas at below freezing temperature is being transmitted. In very cold areas such as northern Greenland it may be sufficient to support utilidors on blocking resting on the natural ground surface. Anchorages in permafrost may be required on steep slopes. Burial under stream beds may involve special problems.

In seasonal frost areas, water lines 6 in. or less in diameter should be laid with invert 6 in. (15 cm) below the computed maximum frost penetration depth. Larger water pipes should be laid so that the top of the pipe is at the computed maximum frost penetration depth. This burial depth can be minimized by the use of frost shields. In areas of very deep seasonal frost penetration or thaw-stable permafrost, it may be more economical to place the entire system of water pipes at nominal depth and provide continuous circulation and heat during the freezing season. Because of wide variation in operating conditions, it is difficult to give a simple rule for minimum depth of sewer pipes to prevent freezing. However, sewer pipes located according to the above criteria for water pipes should always be safe, as sewage leaving a building is normally appreciably warmer than the water supply entering the building.

If utility devices such as underground telephone or power lines are placed in the annual frost zone, they may be damaged by stones that are heaved differentially or by frost cracking of the ground.

Utility line and utilidor guidance is presented in TM 5-818-5[9].

14. CONNECTIONS OF UTILITIES TO BUILDINGS. Care should be taken so that utility lines (fuel, water, sewer, electric, telephone and other) will not be sheared by heave or settlement where they pass through foundation walls and that water and sewer lines will not freeze. When utility lines enter a building in a seasonal frost area below ground level, they should be placed below the depth of seasonal frost penetration. If utility lines must pass through an annual frost zone containing frost-susceptible materials they should be oriented parallel to the direction in which frost heave acts and should be protected by anchored frost isolation sleeves.

15. DRAINAGE AROUND STRUCTURES. In all frost areas, the ground surface adjoining a building should slope away from it and all concentrated surface or roof drainage should be taken several feet away by concrete guttering or other means, to minimize effects of frost action on the foundation. Water from flat roofs which is piped down inside the building should be discharged into the ground below the depth of maximum frost penetration in seasonal frost areas and, if permitted, into the building sewer system in permafrost areas. Melting snow is a common source of undesirable surface water, the effects of which can be partly controlled by good snow removal practices near structures.

Subsurface drainage systems are not usually practical in areas of very deep seasonal frost or permafrost unless they can be placed in ground which will be unfrozen at the time they need to function and outlets onto the surface can remain free from ice blockage. They can be successful at the base of a footing where heat from the building keeps them functional, if suitable discharge is available.

Considerable damage can be caused to permafrost foundations by seemingly insignificant water movement through unfrozen ground near water bodies or above the permafrost, particularly when the flow has originated as surface water exposed to summer heating. Wastewater from buildings, particularly hot water or waste steam condensate, must never be allowed to discharge on or into the ground near a permafrost foundation. Even small amounts such as minor leakage from steam lines can have serious progressive adverse effects. All utility lines carrying steam or liquids must be kept completely tight. Drainage ditches cut into ground underlain by permafrost should be avoided if at all possible. Although cellars and basements are desirable in seasonal frost areas because any heat loss helps to protect against heave they should be avoided under heated buildings on permafrost containing excess ice, not only for foundation stability reasons, but also because the cellar or basement becomes a sump into which any meltwater will drain. TM 5-852-4[8] discusses these questions in more detail.

16. STABILITY OF SLOPES DURING THAW. In both seasonal frost and permafrost areas, slopes composed of fine-grained frost-susceptible soils tend to experience frost sloughing as illustrated in Figure 34a. The sloughing occurs when ice-filled soil thaws relatively rapidly and the resulting wet, low shear strength soil slumps downward. The ice in the soil may have formed by ice segregation during the winter, or in permafrost areas ice-filled soil may have been exposed by excavation (Figure 34b). One of the most useful methods of coping with this problem is slope blanketing with gravel, as illustrated in Figure 34c. Required blanket thickness may range from about 6 to 30 inches (0.15 to 0.76 m) in seasonal frost areas and from 18 inches (0.46 m) up to as much as 5 feet (1.5 m) in permafrost areas. A covering of peat may also be used in permafrost areas to reduce the depth of annual freeze and thaw. It may sometimes be advantageous to make the initial cut in permafrost quite steep and to allow natural degradation and build-up of protective cover of thawed and drained material to occur for up to several years before placing the final protective granular blanket.

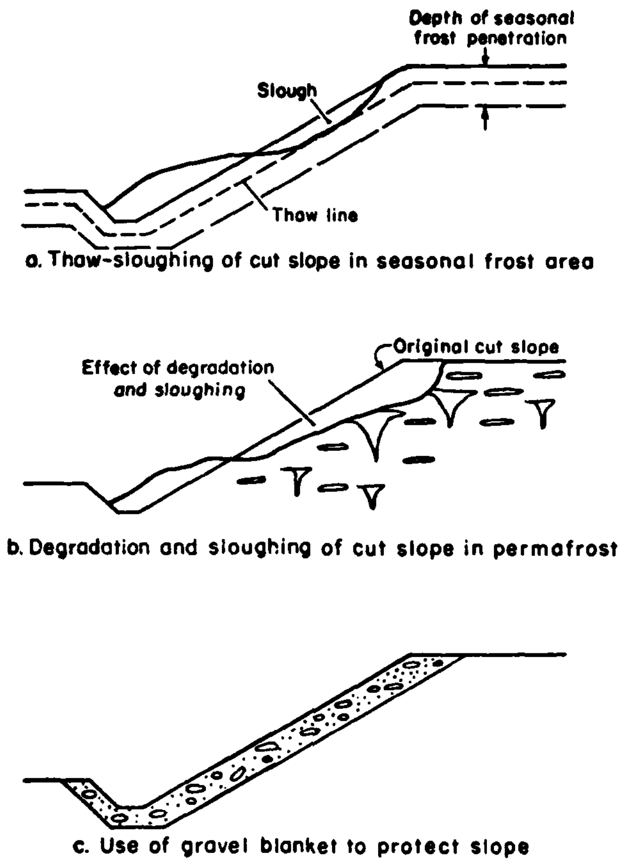

Figure 34. Slopes in seasonal frost and permafrost areas.

Under favorable conditions it may be possible to defer placement of a blanket indefinitely, but the slope may be unsightly and significant maintenance measures may be required because of erosion, drainage problems and possibly sloughing or slides. TM 5-852-4(8) discusses this subject in greater detail.

17. CONSIDERATION OF CONSTRUCTION PROBLEMS. Design decisions should take into account construction factors of special importance in cold regions such as length of construction season, site seasonal accessibility, and kinds of equipment and materials which can be most readily mobilized at the site. Because frozen ground may have the strength properties of lean concrete and because cuts into permafrost may initiate difficult-to-control degradation, requirements for excavation of frozen ground should be minimized or avoided in both seasonal frost and permafrost areas. Successful construction of compacted embankment and backfill under below-freezing conditions may be very difficult to accomplish unless the material consists of gravel drained to very low moisture

content. The length of the season of above-freezing air temperatures during which borrow materials are thawed and can be effectively excavated, transported, placed and compacted should therefore be determined in relation to the types of materials available. Techniques for successful concrete placement under below-freezing air and ground temperatures have been intensively studied and developed, but experience has shown that careful consideration and planning of these details at the design stage is essential. For example, it should not be assumed that concrete formwork can be supported on thaw-susceptible frozen soil thought to be "insulated" by a foot or two of gravel. The heat from warmed air necessary for concrete curing can pass through the gravel in a day or so, and thaw frozen ground unevenly with serious loss of stability and non-uniform settlement of the posts, the forms, and green concrete. The opposite situation can, of course, occur if soil supporting the formwork is allowed to heave. Footings and floor slabs on grade must be carefully protected during freezing periods if frost-susceptible soil underlies the structure within the possible depth of frost penetration (i.e. during construction, when the equivalent of an unheated building condition may exist). Most difficulties from frost action during construction result from neglect of this elementary requirement. Multiple-story buildings have been lifted as much as 6 in. (0.15 m) where this precaution was not exercised. Necessity for such protection should be anticipated during design and covered in the specifications.

REFERENCES

MILITARY STANDARD

1. Unified Soil Classification System for Roads, Airfields, Embankments and Foundations. Military Standard MIL-STD-619B, June 1969. Includes system for frozen soils.

DEPT. OF THE ARMY TECHNICAL MANUALS

2. TM 5-809-10 — Seismic Design for Buildings.

3. TM 5-818-1 — Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures).

3a. EM 1110-345-310 — Foundations Subject to Vibratory Loads.

4. TM 5-840-1 — Cold Storage Facilities.

5. TM 5-852-1 — Arctic and Subarctic Construction, General Provisions.

6. TM 5-852-2 — Arctic and Subarctic Construction, Site Selection and Development.

7. TM 5-852-3 — Arctic and Subarctic Construction, Runway and Road Design.

8. TM 5-852-4* — Arctic and Subarctic Construction, Foundations for Structures.

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DESIGN OF FOUNDATIONS FOR BUILDINGS IN DISCONTINUOUS PERMAFROST

By Arvind Phukan[1]

ABSTRACT

Various foundations may be designed to construct buildings in discontinuous permafrost regions. However, the nature of the thawed-frozen soil conditions is yet to be fully analyzed to optimize the design criteria under several critical conditions. The long term effects may cause severe deformations or differential settlements of foundations causing distress and/or failure of buildings.

This paper illustrates many possible thawed-frozen soils boundaries which may prevail in discontinuous permafrost regions and design considerations are innovated to consider various factors such as thermal effects, physical-mechanical properties of unfrozen-frozen soils, ground water conditions and surface loading conditions. Merits of various foundation designs are evaluated to arrive at a design philosophy which may be used for building foundations in discontinuous permafrost conditions.

INTRODUCTION

Recently, construction activities in the permafrost regions of the world have been increased significantly. Consequently, an expansion of engineering design of foundations for various possible structures is imminent. The cost of over-design for foundations is high as they are expensive to build and install. It is imperative that design philosophy and adequate data be developed for different subsoil conditions to arrive at cost-effective and stable foundations on permafrost regions.

The distribution of permafrost regions in North America, U.S.S.R., and other countries of the world is well known from various publications (Ref. 1 & 2). Fig. 1 shows the distribution of continuous and discontinuous permafrost regions in Alaska. It is well known that the distribution of permafrost in North America is influenced by climatic and terrain factors (Ref. 1). Especially, in the regions of discontinuous permafrost, the boundaries of frozen and unfrozen surfaces are governed by relief, surface vegetation, hydrology, snow cover, subsurface soil type and composition and surface disturbances (natural or man-made).

Generally, foundation design for structures underlain by permafrost involves one of the following active or passive methods:

1) Keeping the soil frozen

2) Allowing the soil to thaw and accepting the resulting settlement

3) Modifying the site by pre-thawing or excavation and replacement with stable materials

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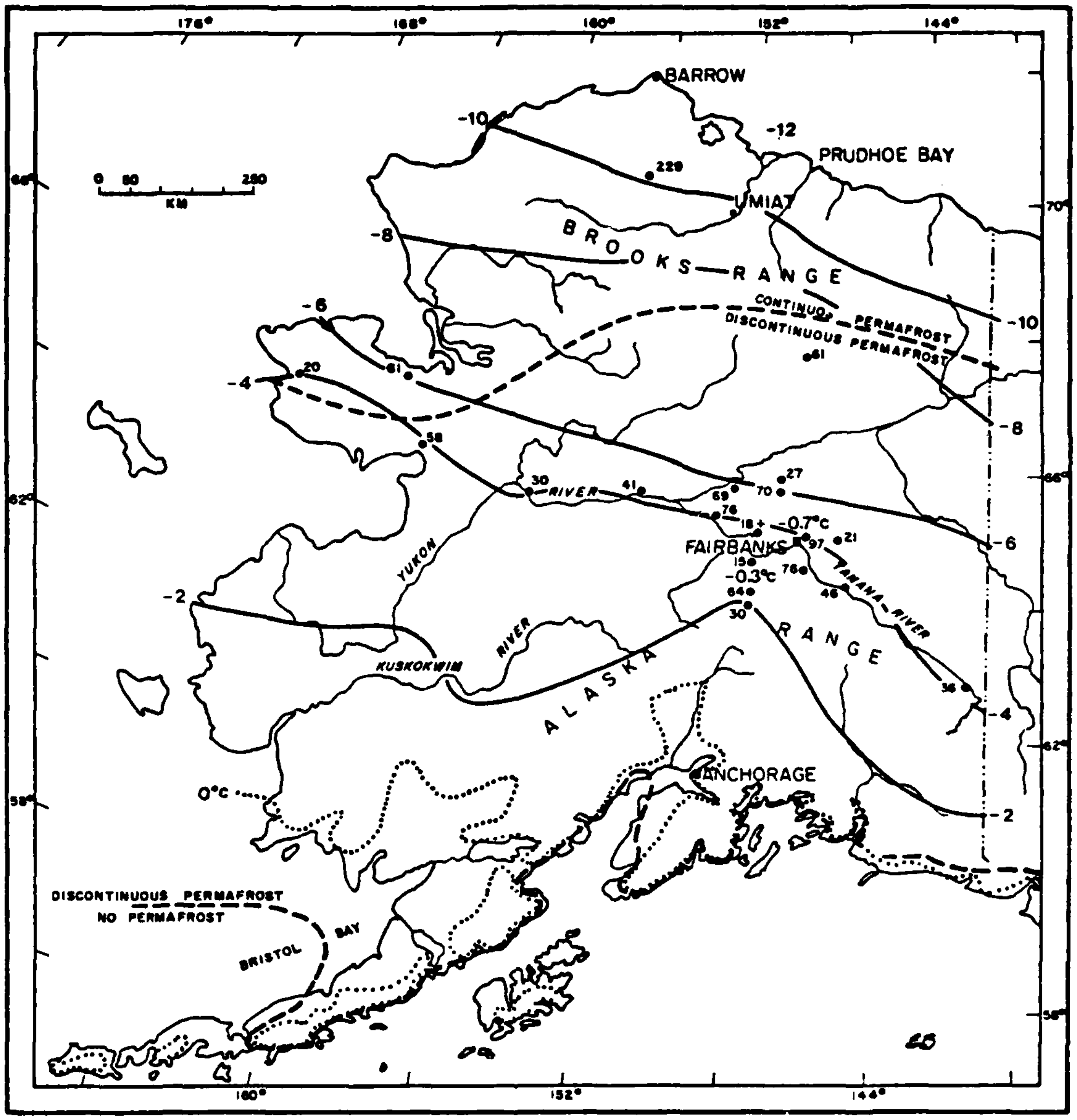


LEGEND

PERMAFROST

122 ● THICKNESS, DEPTH TO BASE (IN METERS)

−6.5°C ● TEMPERATURE OF PERMAFROST AT 15 TO 25 METERS

— — — PERMAFROST ZONE BOUNDARY

CLIMATE

········ APPROXIMATE POSITION OF MEAN ANNUAL AIR TEMP. ISOTHERM, 0°C

——— MEAN ANNUAL AIR TEMPERATURE, °C

Figure 1. Permafrost in Alaska (after Brown and Péwé, Ref. 1).

4) Disregarding the permafrost

A general design approach to foundations in frozen soils is presented in Table 1. Method 1), involving some means of retaining permafrost in the frozen state, is commonly used in both continuous and discontinuous permafrost regions. Method 2) is usually acceptable only if the underlying frozen soils are basically thaw-stable, or if the structure is a temporary one. Method 3) which can be both difficult and costly if the site is underlain by thick deposits of unstable soils is generally applicable to discontinuous permafrost. Method 4) is usually acceptable when the permafrost foundation material consists of sound bedrock or dense thaw-stable soils. Some of the buildings, residential, commercial and industrial, which are built around Fairbanks, encountered severe foundation problems resulting in partial and/or complete rejection and major repair (Figs. 2 to 4). The Fairbanks area is in the discontinuous permafrost region (see Fig. 1) and is the largest Interior Alaskan city. Geologic backgrounds of the area have been studied widely and reported (Ref. 3 & 4). The thickness of permafrost near Fairbanks varies from 33 to 60 m, the mean annual ambient temperature is -3.5°C and the freezing index is in the order of 3,300 degree days.

This paper analyzes different thawed-frozen subsoil boundary conditions as encountered in the discontinuous permafrost regions of Fairbanks. Design aspects of building foundations in such subsurface conditions are discussed to arrive at a design philosophy that may be used for building foundations in discontinuous permafrost.

DISCONTINUOUS PERMAFROST PROFILE

Six different thawed-frozen boundary conditions which may be encountered in the discontinuous permafrost regions are illustrated in Fig. 5. Focusing on the design of different foundations for buildings on these profiles, it will be essential to delineate soil type and composition including ice matrix, ground water conditions, thermal profile with exposure conditions, and soil drainage, if any. Design conditions are complicated by the presence of fine-grained soils in the thawed or frozen zone. The saturated, fine-grained silty soils are favorable to growing vegetation that acts as insulation to minimize the mean annual ground temperature and such conditions aggravate the permafrost configuration. Consequently, significant areas of saturated unfrozen fine-grained silty soils that may occur naturally are remote in the permafrost regions unless external agencies like streams, rivers and relief play a major role to alter the ground temperature regime. A reasonable section containing saturated unfrozen fine-grained silty soils (Ref. Fig. 5(d)) could be found in two areas, (I) active, high energy floodplains and (II) man-use disturbed area containing fine-grained soils continuously maintained for considerable time.

Fig. 5(a) represents generally a shallow active layered section with thick insulating vegetative cover. Such configuration indicates minimal thermal disturbances. Fig. 5(b) indicates a deeper active layer which is possible with minimal surface cover. Figs. 5(c) & 5 (d) are identical with the exception of ground water conditions. Such profiles are possible with the thermal disturbances of natural ground conditions. Fig. 5(e) is generally described as a shallow discontinuous permafrost condition, whereas

FROZEN SOILS

- **Unfavorable Foundation Materials When Thawed** (Fine-grained soils or rock containing ground ice)
  - **Maintain Existing Thermal Regime** (Applies to continuous and discontinuous permafrost zones)
    - Permanent Construction
      1. Piling
      2. Spread footings
      3. Posts and pads
      4. Ducted foundation
      5. Artificial refrigeration
      6. Rigid structural base
    - Temporary Construction
      1. Posts and pads
      2. Sills
      3. Slabs or rafts
  - **Accept Thermal Regime Changes Caused by Construction** (Applies to continous and discontinuous permafrost zones)
    - Permanent Construction
      1. End bearing, piles, caissons, or footings to stable stratum
      2. Rigid structural base (for small structures only)
    - Temporary Construction
      1. Piling
      2. Perimeter sills
  - **Modify Foundation Conditions Before Construction** (Primarily applies to discontinuous permafrost zones)
    - Permanent and Temporary Construction
      - Use designs applicable for conditions resulting after;
        1. Pre-thaw and pre-consolidation of unfavorable materials
        2. Replacement of unfavorable materials
- **Favorable Foundation Materials When Thawed** (Clean, granular soils or rock without ground ice)
  - Use designs applicable to temperate zone

Table 1. Foundation design approach for frozen soils (modified from TM5-852-4).

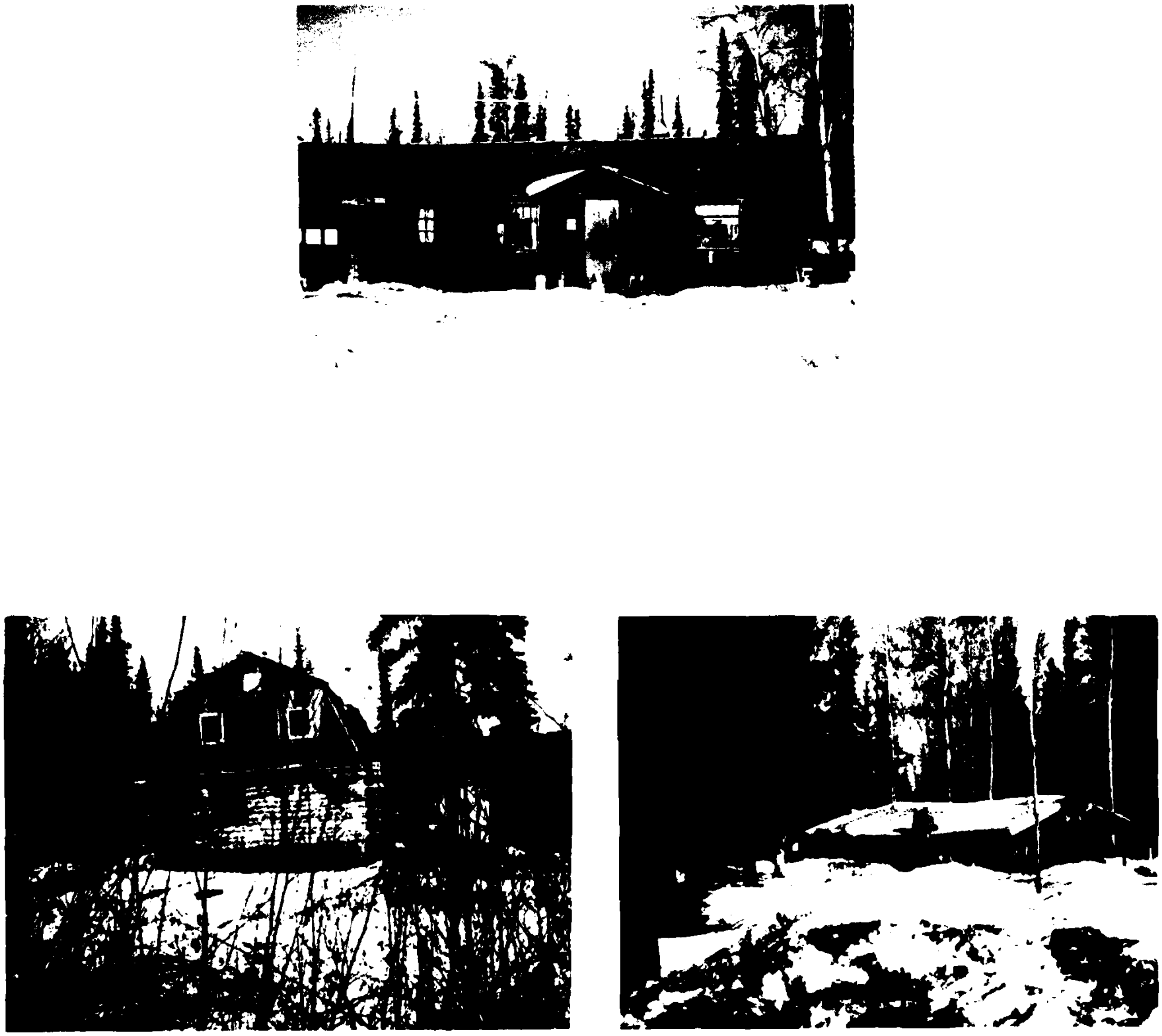


Figure 2. Cases of residential building foundation problems.

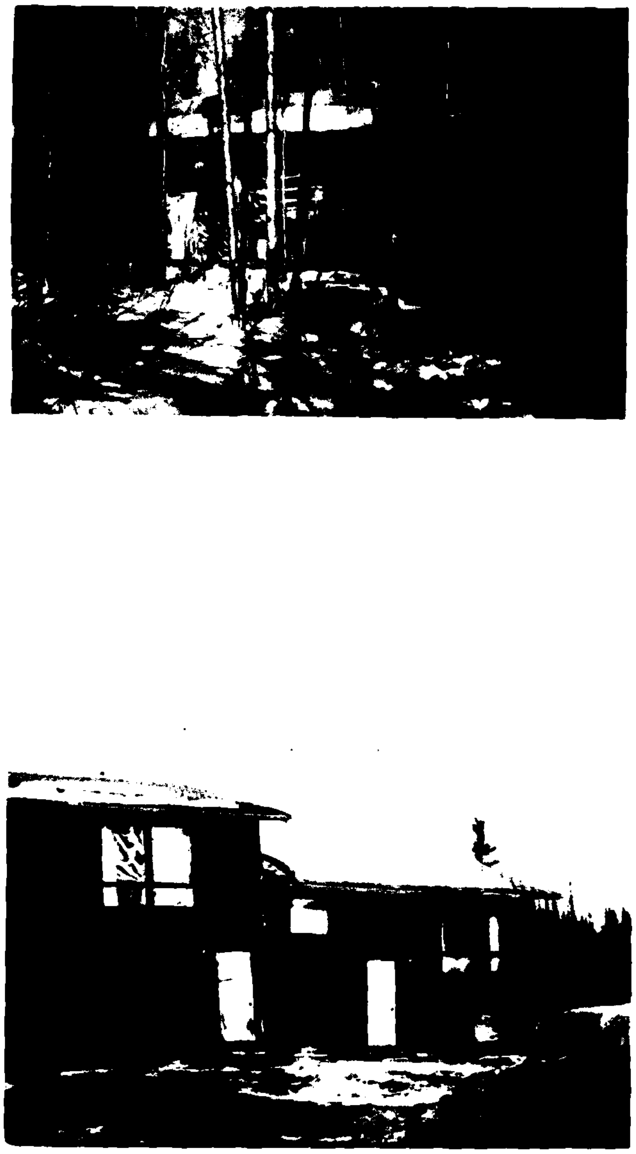


Figure 3. Cases of residential building foundation problems.

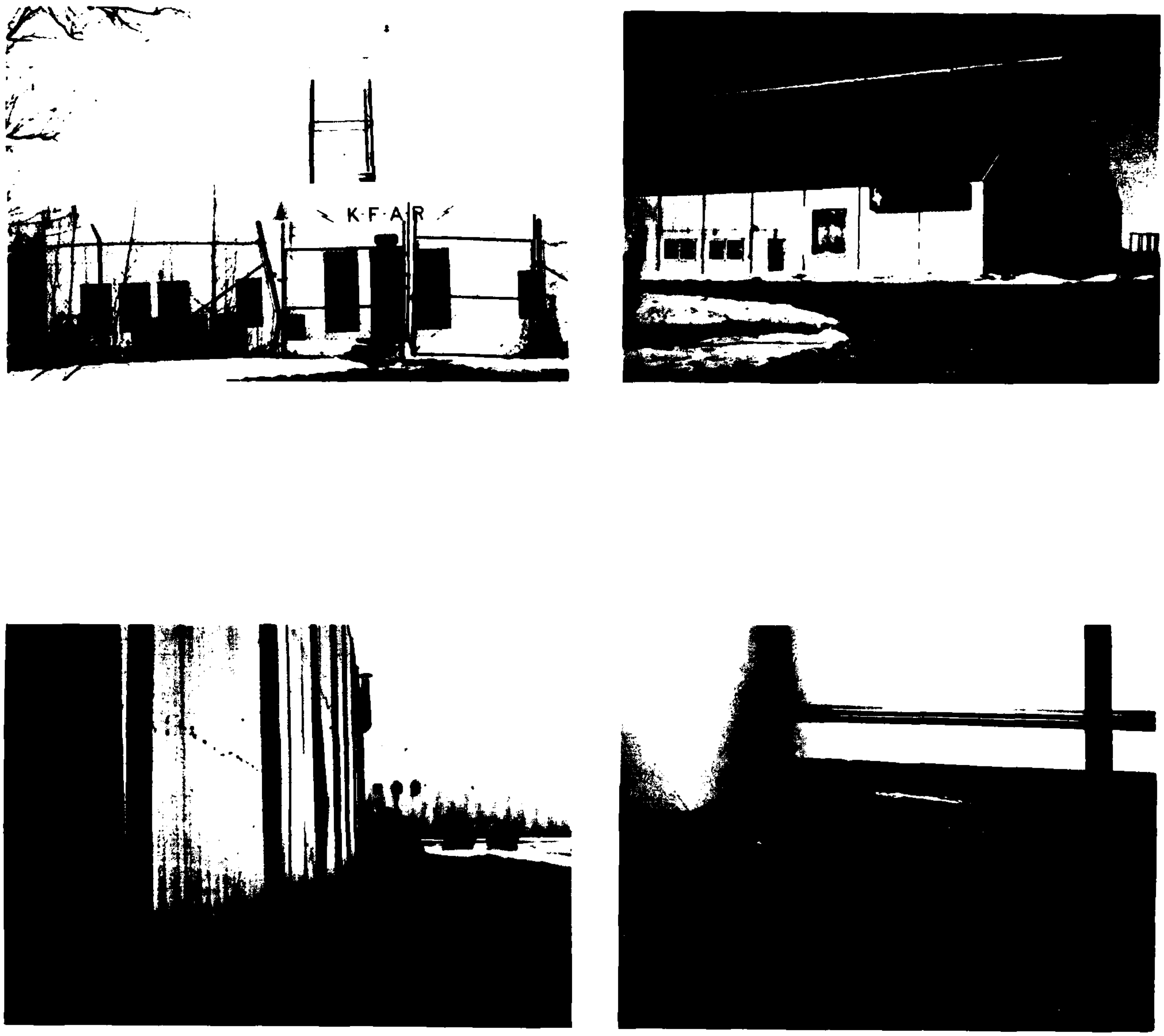


Figure 4. Cases of commercial and industrial building foundation problems.

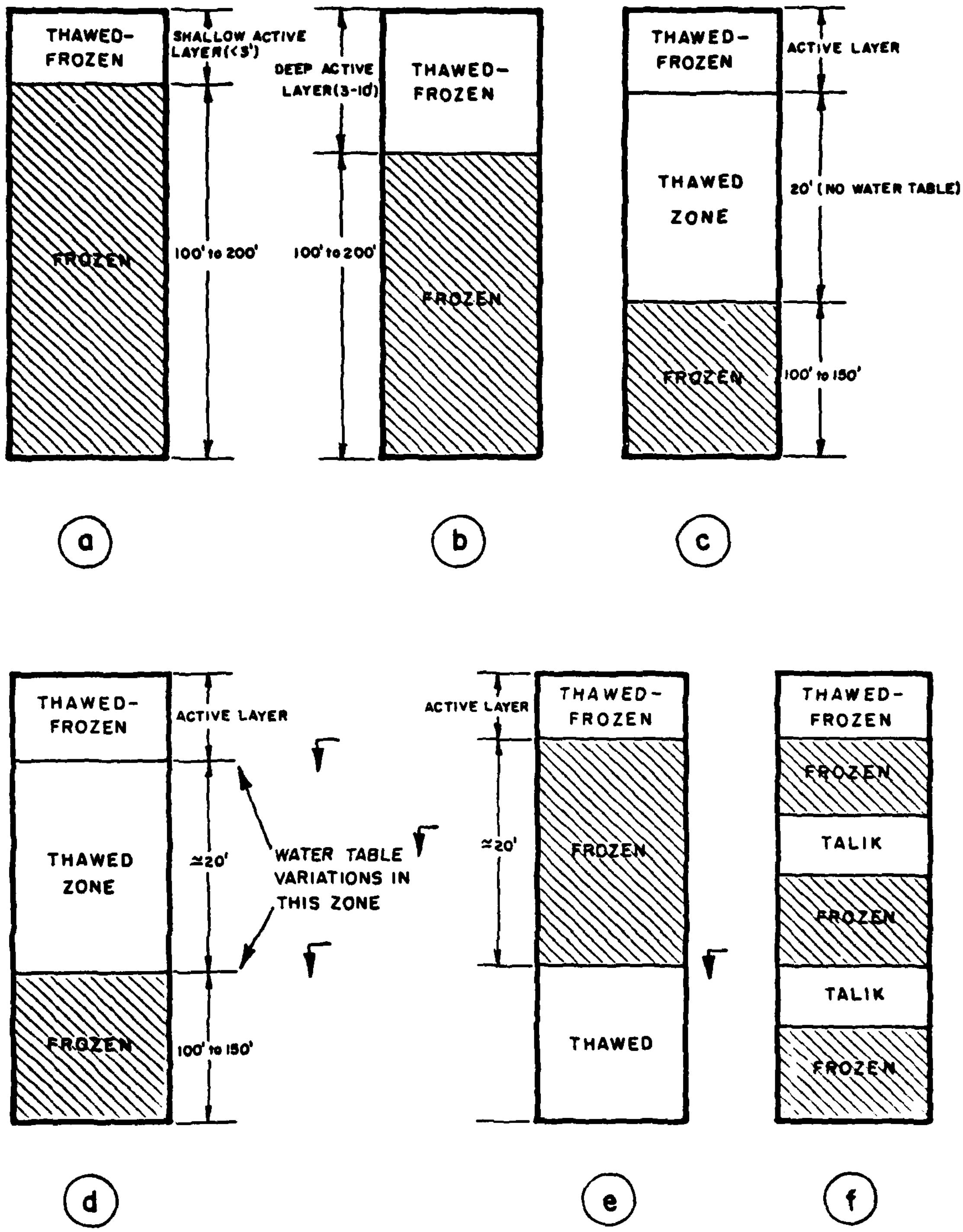


Figure 5. Discontinuous permafrost subsoil configuration.

in Fig. 5(f), the profile is generally common near the active water flow such as river or stream banks in the discontinuous permafrost regions with relatively high ground temperatures near the freezing point.

DESIGN CONSIDERATIONS

Building foundations in discontinuous permafrost regions may proceed in three steps:

1.) Select the most economical type which will provide an adequate factor of safety against failure of the supporting ground under the anticipated loads

2.) Calculate the anticipated deformation or settlement during the design life of the structure. If predicted settlement is in excess of the allowable, the foundation type must be modified and checked again for stability

3.) Determine the feasibility of the proposed building foundation and its construction in relation to factors such as possible thermal degradation of frozen soils, increase in ground temperature, surface drainage and ground water conditions. The thermal interaction between the structure and the ground must be maintained at the level which is needed for the long term stability of the foundation.

The above steps can be included into two broad sections. One deals with the rheological aspects which define the effects of stress upon the foundation material. The other section considers the thermal aspects consisting of heat flow and thermal analysis. Both studies are essential for the stability of foundations. However, engineering judgments must be applied to the ultimate design of safe as well as economical foundations.

The following are the main design considerations for foundations in the discontinuous permafrost:

Active Zone: The seasonal depth of thaw and freezing undergone at the ground surface can be determined by various analytical formulas such as the modified Berggren equation and Stefan equation. Numerical analysis with computer programs can be used for non-homogeneous subsoil conditions. Site conditions are to be considered in determining the depth of thaw or frost penetration and its susceptibility to thermal degradation. The significant surficial vegetative cover and snow cover at the site generally affect the depth of thaw and frost penetration resulting from a higher ground temperature which may exceed the degree days of freezing. The effects of wind in snow drifting must also be considered at the site. Soils with higher water content will prevent thawing from a greater depth than soils with a low water content. However, surface water penetration into the ground will cause a significant increase in heat absorption resulting from a greater thermal degradation of the frozen layer.

Long Term Shear Strength: The shape and type of building foundations in

discontinuous permafrost will be influenced by the long term shear strength of supporting soils. Various investigations (Ref. 5, 6 & 7) have reported adequate theoretical as well as laboratory results for the determination of long term shear strength of soils. Dominant factors to be considered are soil type and composition, temperature range, magnitude of confining pressure and degree of water or ice saturation.

Settlement: A knowledge of stress distribution is required for the determination of settlement. Settlement may be short term or long term (creep) depending on the physical-mechanical properties of in-situ soils, magnitude of loading, and the long term behavior of soils. Various methods (Ref. 8, 9, 10 & 11) are available to predict the short term as well as long term settlement under difficult design conditions.

DESIGN ASPECTS

Design Concept: It is well known that a progressive thaw bulb as well as pressure bulb is produced under a heated building. A typical thaw bulb configuration is presented in Fig. 6. On the other hand, a freeze bulb will be generated under an unheated building where the temperature is lower than the ground temperature. Four design approaches mentioned in the previous section are to be synthesized with the thawed-frozen boundary configuration presented in Fig. 5. The main design concept to be applied is either to limit thaw and thereby prevent degradation of the frozen layer influenced by the structure or to arrest settlement or deformation in the thawed zone, whatever the predominant case.

In order to maintain thaw within the acceptable range, the design concept involves the use of a gravel pad with or without insulation, use of natural or forced ventilated air ducts under the foundation, artificial refrigeration with heat probes or coolant circulation and deep foundations with appropriate heat transfer devices.

Foundation Types: Different foundation types for buildings that can be used in continuous or discontinuous permafrost regions are illustrated in Figs. 9 to 12, inclusive. Conventional type foundations bearing on thick gravel pads have generally been used in the continuous and discontinuous permafrost regions. Insulation can be incorporated into the pad to reduce the required thickness of gravel as illustrated in Fig. 9(b). The gravel pad provides a seasonal heat sink to preserve the frozen state of the underlying natural ground which may be encountered at different depths as shown in Fig. 5. In order to further reduce the required thickness of gravel, which is a scarce and costly commodity in many areas of Alaska, some means of extracting heat beneath the structure can be provided. Air convection systems such as the one shown schematically in Fig. 10 have been used beneath aircraft hangars and maintenance buildings (Ref. 7, 12 & 13). These systems rely on either natural or forced convection of cold winter air through pan ducts in the slab or conduits buried beneath the slab to freeze back during the winter that portion of the gravel pad which thawed during the previous summer. Similarly, mechanical refrigeration systems can be used to circulate chilled fluid through pipes buried beneath the slab to provide year-round heat extraction. Such systems are often used

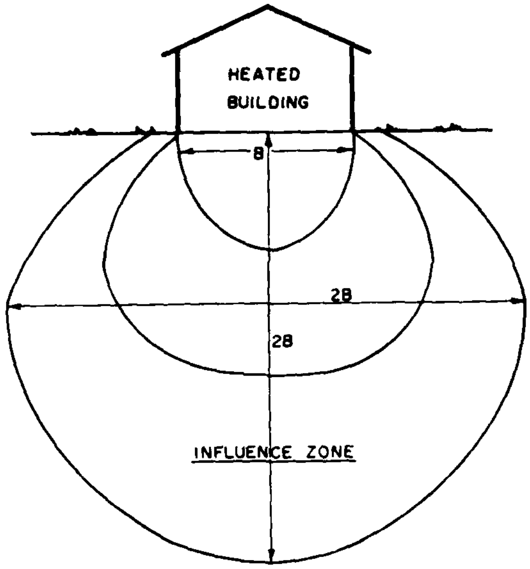


Figure 6. Typical thaw bulb under a heated building.

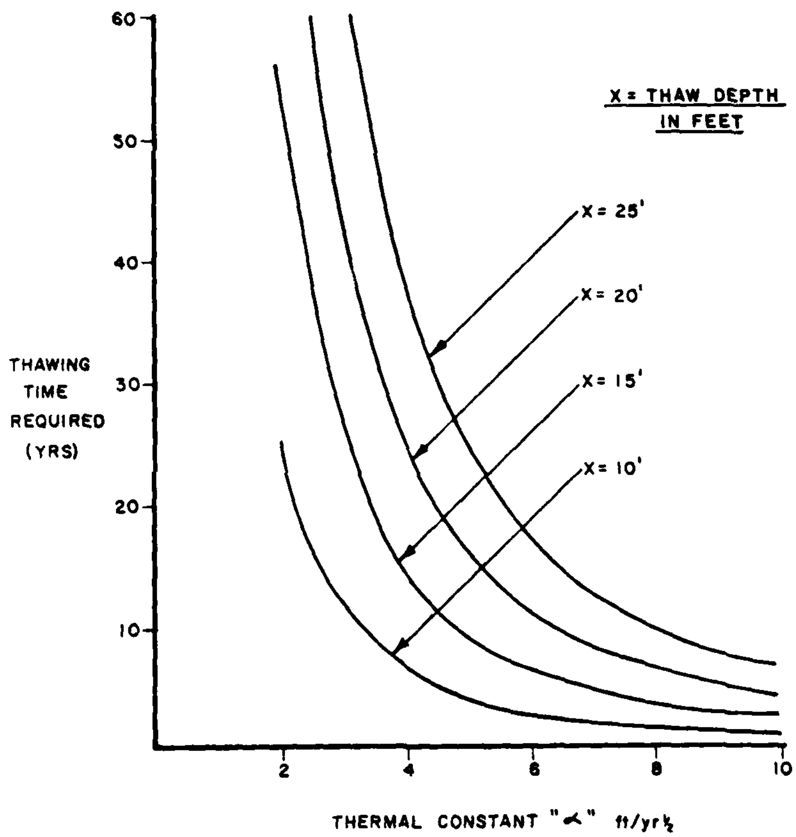


Figure 7. Relationship between thermal constant "α" and thaw penetration.

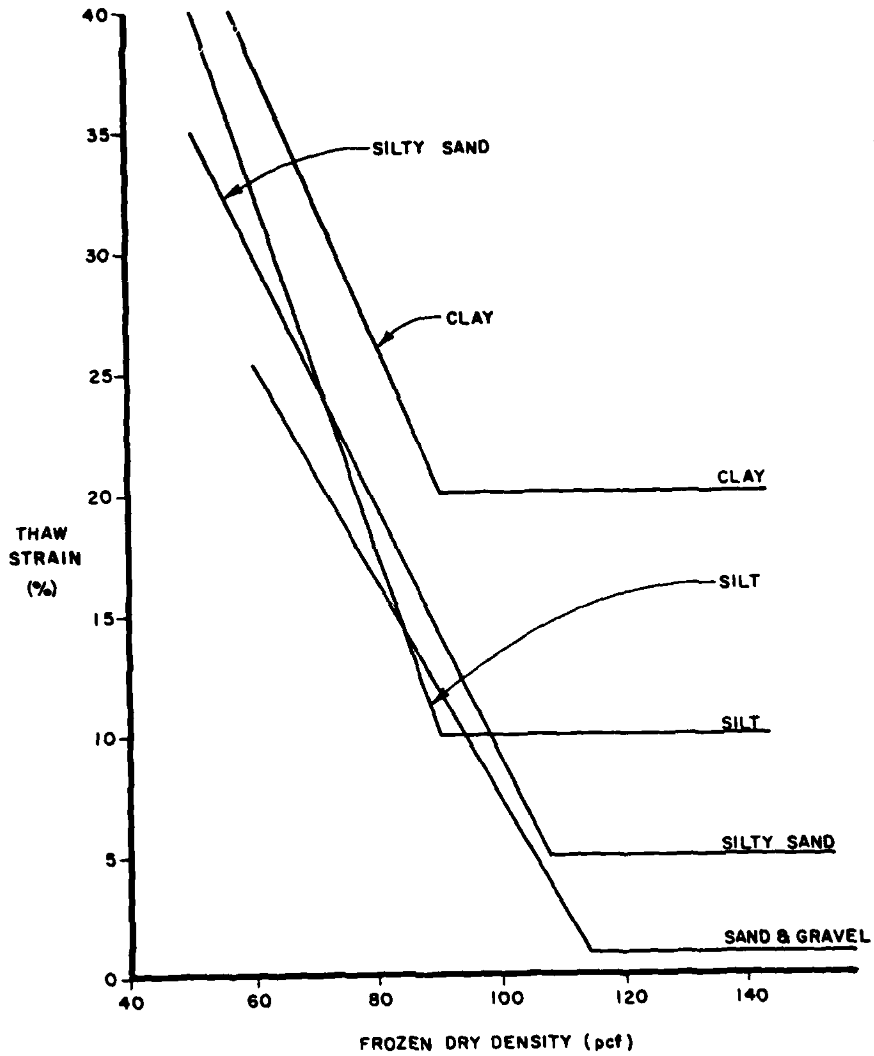
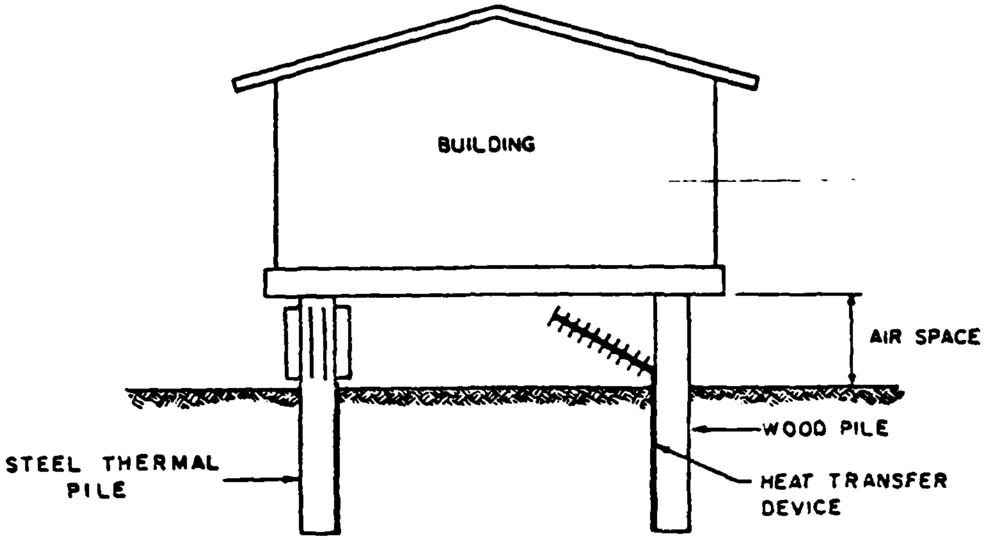
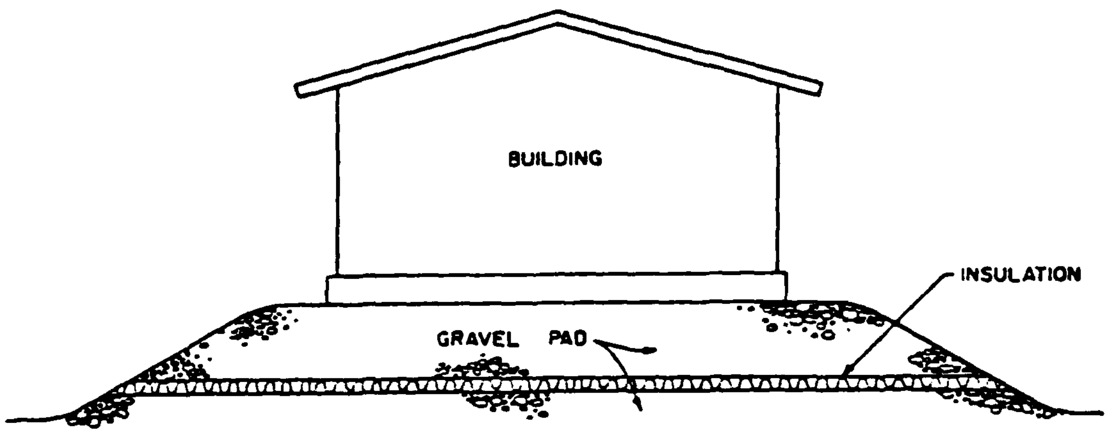


Figure 8. Relationship between dry density and thaw strain for different soils.



(a.) ELEVATED BUILDING WITH THERMAL PILE FOUNDATION.



(b.) BUILDING FOUNDED ON GRAVEL PAD WITH INSULATION.

Figure 9. Typical arctic foundations.

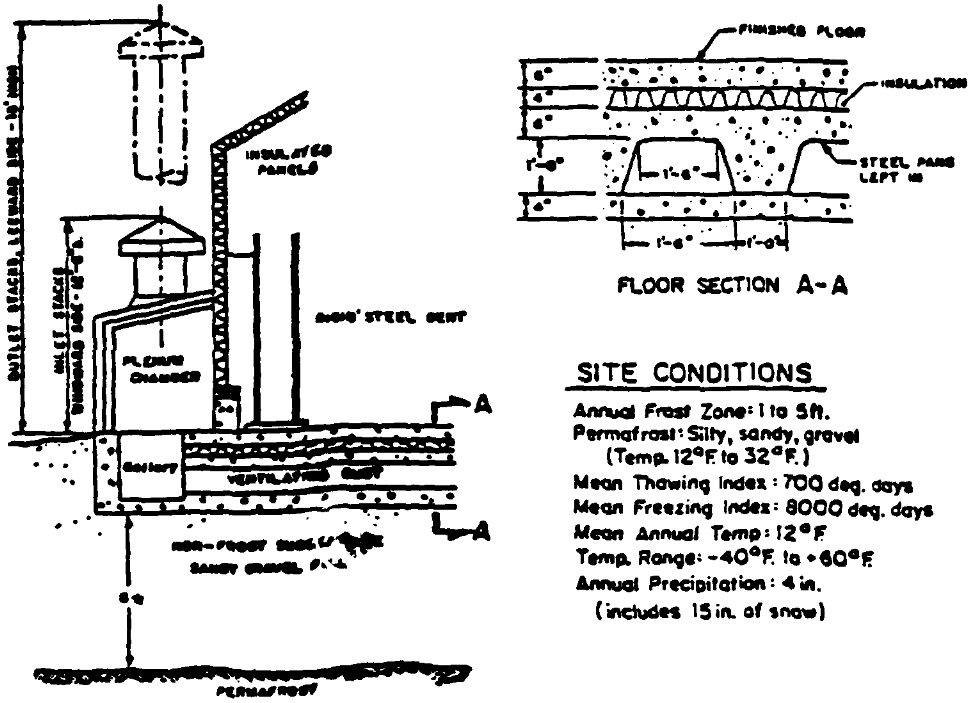


Figure 10. Pan-slab foundation showing plenum chamber and ventilation duct (after Sanger, Ref. 7).

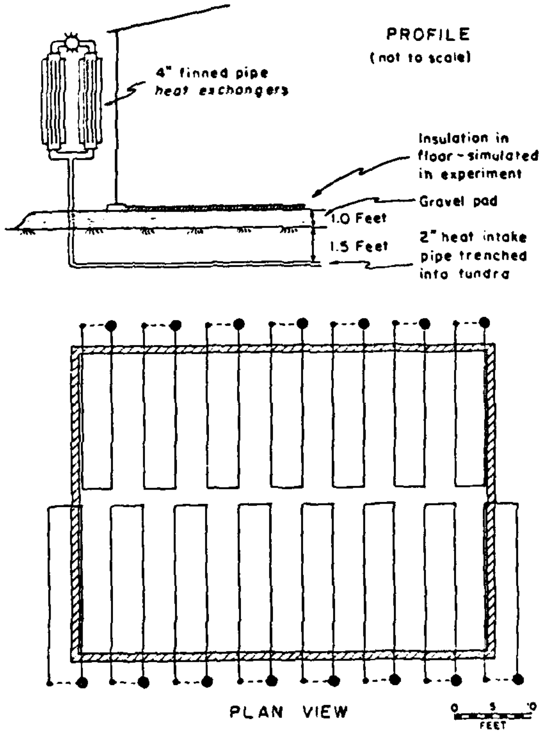


Figure 11. Convection cells-CEL experimental building at Barrow (after Cronin, Ref. 15).

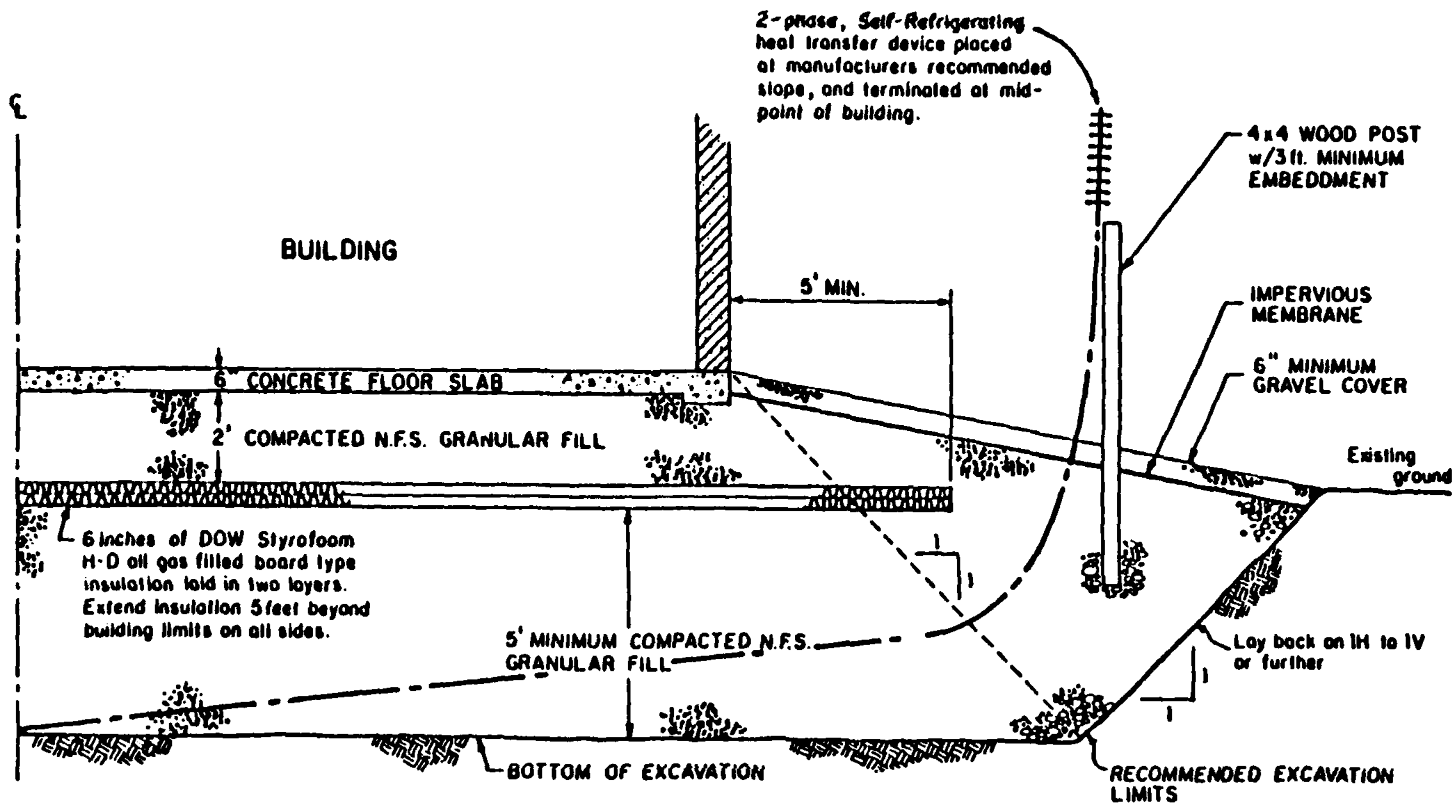


Figure 12. Self-refrigerated insulated gravel pad (after Phukan et al., Ref. 13).

beneath generating plants, crude oil pumping stations, and other structures which impart high heat and vibratory loads. Both systems have drawbacks in terms of required maintenance and reliability, and in the case of mechanical refrigeration and forced air convection systems, the necessity of an external power source.

The use of self-refrigerating heat transfer devices as a means of heat extraction from beneath insulated gravel pad foundations provides a viable foundation design for heavily loaded ground floor buildings (Ref. 13) and a typical cross-section of such a foundation type is shown in Fig. 12.

The U.S. Navy Civil Engineering Laboratory experimental installation used horizontal loop-configured liquid convection cells, as shown in profile and plan in Fig. 11. Loop-configured cells were used because tests had shown that inclined single-ended liquid convection cells perform very poorly (Ref. 14). The objective was to strictly minimize the required gravel pad thickness. Therefore, the loops, constructed of 2-inch pipe with 4-inch finned pipe heat exchangers above ground, were buried in trenches in the ice-rich frozen silts rather than in a gravel pad. Gravel was used only to elevate the building above summer surface water on the tundra. The use of ice-rich silts as a seasonal heat sink has resulted in cyclical heave of about 1 inch at the foundation, and of about 2 inches at the floor, based on the first year's performance (Ref. 15). Although such performance is not ideal, the settlement which would have occurred if the massive ice which is present below the convection cells had thawed could have been spectacular, to say the least.

Elevated, post-and-pad or passive piling foundation systems may be used in the continuous and discontinuous permafrost regions for supporting structures. With this type of foundation system, a free air-space is provided beneath the structure to reduce the influx of heat from the building into the soil and to permit the cold winter air temperatures to cool the bearing soils. In many instances, the combination of the air-space and the shade from the structure is sufficient to limit seasonal thaw and maintain the thermal conditions beneath the structure.

In the discontinuous permafrost region, where the ground temperatures are higher and the depth of seasonal thaw greater, or in the colder, continuous permafrost areas where some large structures impart high heat loads to the underlying soils, self-refrigerating single phase and two phase heat transfer devices are often used to extract heat from the ground during the period of the year when the air temperature is colder than the ground temperature. A typical elevated building with thermal piles is shown in Fig. 9(a). Single phase devices are liquid filled, and rely on a natural convection flow set up by the difference between ground and air temperature to extract heat from the ground. The liquid is warmed by the heat in the ground, rises to the upper (heat exchanger) portion of the device where the heat is rejected to the cold ambient air, and then sinks to the lower portion of the device, where the process repeats itself. The two phase device is pressurized with a liquid/gas system. The liquid in the lower portion of the device is vaporized by the heat in the ground, and the gas rises to the heat exchanger, where it condenses, rejecting heat to the surrounding air. The condensed liquid then flows to the bottom of the

device to repeat the cycle.

Method of Analysis: The depth of thaw in frozen soils depends upon the surface temperature, thermal properties of the soil mass, the average initial temperature of the soil at the start of the thawing season, and so on. The modified Berggren Equation (Ref. 10) is most commonly used to determine the depth of thaw penetration in a homogeneous soil mass, and the same method may be applied to non-homogeneous soils by determining that portion of the surface thawing index required to penetrate each layer. The sum of the thickness of all thawed layers is the depth of thaw. The depth of thaw penetration is given by:

$$X = \alpha \sqrt{t}, \tag{1}$$

where: X = depth of thaw, ft.
α = thermal coefficient, ft./yr.$^{1/2}$
t = time, years

The thermal coefficient "α" is governed by the thermal properties of soils, surface temperature or thawing index and the physical properties of soils. The relationship between the thermal coefficient "α" and thaw penetration is illustrated in Fig. 7. This relationship may be used to determine the time required to penetrate different thawed zones.

To calculate the predicted primary settlement, the relationship between thaw strain and frozen dry density as shown in Fig. 8 for different soils may be used. Other analytical relationships mentioned in the previous section may be applied to calculate the time dependent settlement or creep settlement.

Appropriate bearing capacity formulas (Ref. 10) may be used to determine the size of foundations for buildings in the discontinuous permafrost regions. Some investigators (Ref. 5 & 8) have used semi-empirical relationships for solving bearing capacity problems by substituting the time and temperature-dependent strength and deformation parameters of the frozen soil into the appropriate formulas borrowed from bearing capacity theories for unfrozen soils.

DISCUSSION AND CONCLUSIONS

Many residential, commercial and industrial buildings founded in the discontinuous permafrost areas near Fairbanks suffered severe settlements. These settlements are primarily due to:

- heat influx to underlying frozen soils

- inappropriate foundation designs

- insufficient investigation of subsoil conditions

The soil conditions near the buildings illustrated in Figs. 2 to 4 consist of either thawed silt underlain by ice-rich frozen silt or peat underlain by frozen sand and silt (Ref. 17). These profiles are similar to sections shown in Figs. 5(c) and 5(d). It is obvious from the profiles that the total or differential settlement of buildings was caused by the thermal degradation of underlying ice-rich frozen soils. Also, the settlement of relatively compressible soils above the frozen layer due to insufficient foundation design caused a different magnitude of settlements. It was reported that the buildings suffered settlements after two to six years of construction. These examples show the time-dependent thaw penetration effects. The thermal coefficient "α" for silts encountered near Fairbanks is generally found to be 4 to 6 ft./yr.(1/2). Unless proper design is done to arrest influx of heat to the underlying soils in discontinuous permafrost regions, settlement problems will be encountered and these magnitudes can be envisioned by the use of Figs. 7 and 8.

Thus, it can be concluded that:

- subsurface soil profiles must be properly delineated in the discontinuous permafrost regions

- heat flux from the building to the underlying sensitive soils must be accounted for to design different foundation types

- different foundation types for various buildings in discontinuous permafrost regions may be designed, but they must be interfaced with the subsurface conditions and thermal analyses

- the design philosophy presented shows that the primary causes of distress are the thermal degradation and associated deformations or settlements.

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FOUNDATIONS ON COMPLEX PERMAFROST SOILS

By Y.J. Velli[1]

INTRODUCTION

The intensive utilization of regions of the Far North has led to the necessity for developing extensive territories with complex permafrost conditions. These include: thermokarst and subsidence formations, topographical forms due to nivation and solifluction, territories with the presence of heaving hillocks, ice crusts, frost cracks, and other formations. However, saline and highly icy soils are particularly widespread.

SOIL MORPHOLOGY

Salinization of permanently frozen soils takes place everywhere on the Arctic coast and in the valleys of Siberian rivers. It is found also in other districts of the Magadan region, Yakutsk ASSR, Krasnoyarsk territory, and along the construction route of the Baikal-Amur pipeline. The conditions for the appearance of soil salinization in coastal and continental zones are different: on the coast this is the result of transgressions of the sea, and on the continent it is the result of the evaporation of water, atmospheric precipitation, surface runoff, and ground water. Correspondingly, the compositions of the salt in these territories and their distribution with depth are not identical. Thus, calcium and magnesium cations predominate in the chemical composition of the salts of Yakutsk, and sodium cations predominate on the Arctic coast. The salinity of the surface layer, as a rule, amounts to 0.2-0.4% of the dry weight of the soil, and it increases with depth, reaching a value of 0.6-2.0% at a depth of 3-5 m.

Highly icy soils (containing inclusions of ice in an amount of more than 40% by volume) are found in practically all permafrost regions. A particularly great amount of highly icy soils are found on the Chukot, Taymyr, and Yamal peninsulas, and on the Arctic coast.

Subsidence forms of topography (thermokarst formations) may appear, upon the thawing of ice, in a section with ice lenses and inclusions close to the upper level of permanently frozen soils with an increase in heat flow into the soil during the summer period, as a result of a change of the hydrological conditions, or an increase in the heat conductivity in the upper layer, or the destruction, as a result of humid activity or other reasons. A distinguishing feature of thermokarst is the progressive increase in sinkholes, accompanied by land slip along shores and depressions. Lakes, the depths of which reach 1.5-3.0 m, appear. Sometimes, they extend over

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extensive territories, forming swampy lowlands. Erosional processes, leading to the formation of ravines, develop when icy soils are washed with runoff water. This process may proceed very rapidly. The thawing of the ice cementing soil particles leads to the destruction of the soil structure. The soil particles are easily removed by water, which leads to the thawing of new layers of soil. The destruction encompasses new sections and may stop only in the case of a change favoring this process, for example, upon the cessation of runoff activity.

FOUNDATIONS ON SALINE SOILS

Both saline and highly icy soils are characterized by a low mineral capacity. Therefore, in construction practice, deformations of buildings and structures raised on these soils are frequent, especially when heat enters the soil. The causes of the deformations are different. The most frequent and characteristic cause for the increase in soil temperature is the unsatisfactory construction of cellars. We may mention here how construction of cellars can be so:

1) An incorrect choice of the type of cellar, for example, the use of a cellar ventilated through an opening at the surface of the ground, in snow drifting and light wind regions.

2) An incorrect orientation of buildings with respect to the snow depth and the local topography.

3) An incorrect choice (without calculation) of the height of the cellar, the number and dimensions of air holes, the distances between them, and the arrangement thereof along the faces of the buildings.

4) Errors in setting up supplementary measures for cooling foundations.

An incorrect use of cellars, cluttering them, covering air holes during the winter period, setting up obstacles in snow drifting paths, and so forth.

THERMOKARST FORMATION DUE TO CONSTRUCTION

The deformations are manifested in the form of settling of foundations, the formation of cracks inside buildings, sagging of the ridge of the roof, and so forth. Deformations develop particularly intensively in the case of the progressive thawing of foundations. As a rule, this is connected with the absence of ventilation of cellars: the zero isotherm moves downward, a thawing cup develops, ice lenses and interlayers thaw, and significant settlings of foundations arise. Most often, deformations of this type are manifested in buildings with increased sources of heat liberation (boiler rooms, diesel engines, baths and laundries, kitchens, and so forth). Particular attention should be given to the heating effect of waters; these include: surface, ground, industrial, or from sewers entering the foundation of buildings. They act periodically although very intensively in time. Soaking into a foundation, water, especially if it is of high temperature,

may cause local thawing of the soil for short periods and for a significant depth. Often in the case of an accident, water passes along a duct, washing out backfilling and causing settling of heating lines and hot water lines. For the same reason, settlings often arise on roads at the locations of water-carrying pipes. The danger of this type of deformation is that it is impossible to predict the location and development thereof ahead of time; it takes place rapidly, and therefore the consequences may prove to be serious.

Quite often, a change in permafrost conditions in a construction area, connected, as a rule, with human activity disturbing the natural interaction maintained before the development of the territory, is a cause of deformations. The basic causes of deformations of this category are the following:

1) Inundation of a territory as a result of a disturbance of the water runoff conditions in the area (the water falls into the foundation, causing an increase in the ground temperature).

2) The formation of interpermafrost pressurized water levels as a result of the freezing of the thawing cup formed under a building in the period of utilization (water broke through and flooded the territory).

3) Thermokarst formations, in which an area may be converted into a mash of liquefied soil with a moving mud flow during a short period of time. These formations may arise when setting up poles, laying cable lines, and so forth, when the ground is insufficiently compacted after a structure is built and the peat-moss cover is not restored. Ravines may form upon the erosion of the upper layer by water.

The causes of deformations of buildings and structures discussed usually are characteristic in the case of construction on saline and highly icy soils.

This hypothesis explaining the cause of building deformations by the lowered bearing capacity of saline soils was put forward in 1958.

FOUNDATION DEFORMATIONS ON ICY AND SALINE SOILS

A frozen saline soil is a complex aggregate consisting of mineral particles of crystals of pure ice, brine, gas bubbles, and salt crystals, under specific temperature and moisture conditions. As research conducted in underground chambers in the Andermin Permafrost Laboratory showed, the freezing temperatures of saline soils always are lower than zero, and depend primarily on the soil moisture, the salinity level, and the salt composition. With an increase in the salinity from 0.25-1.5% the freezing temperature decreases as illustrated below:

with a soil moisture of 20%, from -0.6 to -4°C;
with a soil moisture of 25%, from -0.4 to -3.2°C;
with a soil moisture of 35%, from -0.2 to -2.6°C;
with a soil moisture of 45%, from -0.1 to -2.0°C.

A significant compressibility of frozen saline soils was observed in conducting compression tests. At a temperature of -1.5°C with an increase in salinity to 0.5% the indicators of compressibility increased by 4 and more times in comparison with frozen nonsaline soils. The compressibility of frozen saline soils also was confirmed by observations of building settling. The settling of the basement of a residential building in Amderma Village, with a soil temperature of -2.4°C and a salinity of 0.3-1.0%, during the summer reached 9 cm.

The compressibility of frozen saline soils is connected with the unfrozen water contained in the soil. In the case of a low salt content in the core solution, the unfrozen liquid phase will consist of bound water and a salt solution which does not freeze at a given temperature and concentration. In the case of a higher salt content, the unfrozen phase will be only the firmly bound water and the salt solution with an equilibrium concentration. At the freezing temperature of a saline soil, not the entire salt solution, but only part of it, freezes in the soil. With a further lowering of the temperature there is a gradual freezing of the core solution with the liberation of pure ice and an increase in the salt content of the remaining water. The amount of unfrozen water increases with an increase in the salinity of the ground. Thus, in loams of temperatures of -2.8 to -11°C, a change in the salinity of the soil from 0 to 1% led to an increase in the amount of unfrozen water by 2-3 times.

The strength of frozen saline soils is characterized, in the first place, by the values of the resistances to normal pressure (R) and shear around the lateral surface of freezing (R_{sh}) with the foundation.

The value (R) is found according to formulas containing the maximum longterm values of the equivalent cohesion (C_{eq}) (taking account of both the cohesion proper and the internal friction of the soil) determined experimentally. It was found that the value of C_{eq} of frozen saline soils depends on the same factors as in the case of nonsaline soils, namely the temperature of the soil, the composition and moisture thereof, and the period of action of a load. In addition, the salinity of the soil has primary significance. Thus, in loam soils at a temperature of -1°C, the value of C_{eq} dropped from 0.15-0.28 MPa (in the case of non-saline soils), to 0.05-0.07 MPa with a soil salinity of 0.02-0.25%, and to 0.02 MPa in the case of a salinity of 1.0%. In other temperature intervals, and in other soils, there is a similar picture. In addition, the C_{eq} of sandy soils in natural position (Z = 0.5-0.1%) is lower than the C_{eq} of ideally nonsaline sandy soils (Z = 0%) by 1.5 to 4.0 times. In the case of an increase in the salinity of the sands to 0.5% the bearing capacity thereof sharply drops, and the value of C_{eq} drops; in the case of a temperature of from -3 to -4°C, to 0.3-0.5 MPa (where Z = 0%) to 0.02 - 0.06 MPa. The influence of the moisture of a frozen soil is less intensive than the temperature of the soil and the salinity. With an increase in the moisture from 0.25 to 0.55% the value of C_{eq} dropped by 25-30%.

The soil salinity is the decisive factor determining the value of the maximum long term resistance to shear of saline soils along the lateral

freezing surfaces (R_{sh}). Thus, in powdery loams, an increase in the salt content of the soil to 0.5% and a temperature of -4.5°C led to the reduction of almost two times (from 0.27 to 0.14 MPa) in the value of R_{sh}. With a further increase in salinity to 1.0% the value of R_{sh} was three times less than the original value (0.08 MPa), and where Z was greater than 1.5% there was practically no freezing of the ground with the foundation. A similar picture also was observed in the case of other soil temperatures. With a decrease in the temperature, the value of R_{sh} increases noticeably.

The value of R_{sh} varies substantially depending on the state of the foundation surface. Experimental data shows a 20-30% decrease in the value of R_{sh} on metal surfaces in comparison with concrete surfaces. These discrepancies are caused by the finish of the foundation surfaces. The technique of sinking pilings into frozen soils also influences the value of R_{sh}. The values of R_{sh} in the case of sinking pilings into predrilled holes of large diameter, filled with slurry, are close to standard. In the case of sinking pilings by steaming, the value of R_{sh} proved to be approximately 10% less, which is explained by the vibration of water from the freezing front upon the subsequent freezing of the soil, and, in the final analysis, by the smaller amount of water (ice) in contact with the piling. In the case of forcing piling into a hole of smaller diameter than the piling itself, the value of R_{sh} proves to be almost 30% higher than the standard, which is indicative of the progressive nature of this method of sinking pilings.

Some values of the calculated resistances of frozen saline soils to normal pressure and shear along the lateral surface of freezing with foundation surfaces, that have been obtained on the basis of experimental data and testify to the low bearing capacity of the soil, are presented in Tables 1 and 2.

Investigations of highly icy soils showed that they have a relatively low value of the damped creep limit and a fairly developed linear creep section, the upper limit of which practically coincides with the long term strength, corresponding to the standard period of structural use. Construction practice and calculations show that the settling of structures on highly icy soils may be significant and requires the development of special measures for using the soils as foundations for buildings and structures.

CONTROLLED FILL TO ELIMINATE DEFORMATIONS

The use of dirt fill on the surface of the ground is a very common (and sometimes the only, if underground ice is located close to the surface) solution. Dirt fill makes it possible to reduce the depth of foundations, to eliminate the danger of their swelling with soil, to improve the moisture conditions of the foundation as the result of providing for drainage of rain- and melt-waters, and to use the bearing capacity of the soils of a seasonally thawing layer, having sufficient compaction, in addition to the compression produced during placement.

Table 1.

Calculated Resistances of Frozen Saline Soils to Normal Pressure R

Name of Soil	Soil Salinization %	R_{sh} (kg/cm^2) with ground temperature of -1°C	-2°C	-3°C	-4°C
Powdery sands	0.05	6.0	13.0	16.0	18.0
	0.10	3.0	5.0	9.0	13.0
	0.20	X	2.5	5.5	6.5
	0.50	X	1.5	2.0	3.0
Fine and medium sands	0.10	8.0	12.0	14.0	17.0
	0.20	4.0	8.0	11.0	14.0
	0.50	X	4.0	6.0	8.0
	0.75	X	X	3.5	4.5
Sandy loams	0.20	5.0	7.5	13.0	15.0
	0.50	X	4.0	7.0	9.0
	0.75	X	X	2.0	3.0
Loams and clays	0.20	4.5	6.5	10.0	12.0
	0.50	2.5	3.5	6.5	9.5
	1.00	1.5	2.2	3.0	5.0

X = Soils may be found in the unfrozen state.

Table 2.

Calculated Resistances of Frozen Saline Soils to Shear Along the Lateral Surface of Freezing R_{sh} with Concrete and Wooden Surfaces

Name of Soil	Soil Salinization %	R_{sh} (kg/cm^2) with ground temperature of -1°C	-2°C	-3°C	-4°C
Powdery sands	0.05	0.7	1.2	1.7	2.4
	0.10	0.4	0.8	1.2	1.5
	0.20	-	0.4	0.6	0.8
	0.50	-	-	0.2	0.4
Fine and medium sands	0.10	0.8	1.5	2.0	2.5
	0.20	0.6	1.2	1.8	2.3
	0.50	-	0.6	0.8	1.3
	0.75	-	-	0.4	0.6
Sandy loam, loam, clay	0.20	0.7	1.1	1.4	1.7
	0.50	0.4	0.7	1.0	1.3
	1.00	-	0.3	0.5	0.7

The dirt fill may be made both for individually standing and groups of buildings. In the case of vertical planning, solved by dirt fill, it is possible even to completely raise the entire territory of a development or to use an island solution, in which the dirt fill is arranged only under buildings or structures including roads. Solid filling of a territory is preferable. It simplifies the arrangement of sewers and water drainage systems, and eliminates the formation of closed low sections, inherent to the island solution. In addition, in a number of cases, solid filling of a territory is less economical. The island solution is recommended, primarily, for individually standing buildings.

Special requirements are imposed on the material of fills: the soil should be unswollen, not subject to rapid weathering, easily dug in borrow pits and easily compacted in the fill, and should retain free flowing properties at negative temperatures. Therefore, large fragmented soils and coarse to medium sands with a moisture content less than or close to the maximum molecular moisture capacity are used. Sometimes, inexpensive slags and clinkers are acquired from on-site cement manufacture.

Shallow column foundations often are used in constructing buildings on fills. The height of the fill and also the dimensions of the foundations and the depth of their location are determined by a calculation according to the technique presented in the Reference on Standard Documents. The slopes of the fill are taken to be not less than: A. for large fragment soils - 1:1.5, B. for sands -1:1.75, C. for slags, and D. other similar materials - 1:2. On the side of the fill having the greatest solar radiation, the slope is insulated with turf, boards, and so forth. In the strips where surface or subsurface waters run off, it is necessary to prohibit them from penetrating into the body of the fill with the use of an arrangement of permafrost rollers, waterproof diaphragms, and drainage ditches. During the winter before making the embankments the snow and ice must be removed from the surface of the soil by means of artificial heating of the surface. For practical considerations, the width of a berm along a building must be no less than 2 m and, if it is specified in the plan of organization of the work, provide for the possibility of moving cranes and trucks along the building.

MAINTAINING SOIL STABILITY

When it is necessary to maintain the soil of foundations in the frozen state, most often it is more economical to combine the arrangement of fill under a building with other measures (insulation, the arrangement of a cellar, an unheated story, air and artificial cooling systems) (Fig. 1).

The arrangement of an insulating layer between buildings and embankments (Fig. 1a) imposes a limitation on the development of a thawing cup to a level below which ice inclusions are found. It is necessary to take account of the fact that in the case of inundation insulating cushions may not play their role. The solution with fill insulation for stock, prefabricated collapsible buildings, the period of service of which in one place is short, is promising.

Fills in combination with a ventilated cellar or cold story (Fig. 1, b and c) are very common. They are used primarily under buildings, located on top of ice, in territories subject to thermokarst formation. This solution makes it possible to prevent destruction of a foundation of a building during its construction and utilization, and also to increase the bearing capacity of the soil by means of lowering its temperature as a result of the movement of the upper boundary of permafrost to the surface.

Ventilated fills (Fig. 1d) are made of large fragmented materials (rubble or gravel) creating communicating macropores. The movement of air through the body of the fill is begun and maintained with the use of a ventilating system. A drawback of the solution is the possibility of the formation of a condensate in the fill, filling the pores with ice or dirt, which in the final analysis may lead to cessation of operation of the system.

Fills with air cooling pipes (Fig. 1e) are used, primarily for heat liberating buildings of significant dimensions (power plants, hangars, and so forth), where it is difficult to provide for given temperature conditions by other means, in the case of the presence of highly icy soils in the foundation. The pipes, as a rule, are laid at the base of the foundations, within the limits of the embankment layer, and run into the cellar or around the walls of the building. Cooling of the foundation is achieved by circulating cold external air during the winter period. Arrangements for checking the condition of the cavities of the pipes and cleaning them should be provided.

Fills with cooling of the foundation by a system of pipes with liquid coolants (Fig. 1f) are used in those cases where winter cooling does not ensure preservation of the foundation in the frozen state. It is particularly effective on saline soils. For the most part, freezing is performed with the use of ammonia or freon refrigerators. The operating efficiency of a cooling system may be increased with the use of natural cooling during the winter in order to lower the temperature of the brine, and also with an increase in the thermal resistance of the floor construction. Continuous operation of the system during the year is then possible. With time the danger of the pipes failing and brine entering the soil, causing salinization of the material with loss of the bearing capacity, increases. Consequently, as soon as stable freezing is ensured it is necessary to move to other methods of maintaining the ground in the frozen condition, to remove the coolant from the pipes, and to conserve the cooling setup.

LIGHTWEIGHT STRUCTURAL USE TO MINIMIZE SOIL DEFORMATIONS

In addition to efforts of strengthening the foundations of buildings, there is another trend: to change the structural members of buildings in such a way as to obtain the possibility of raising them in complex permafrost conditions without supplementary measures to reinforce the foundation or in combination with them. This is the method of creating light, fully prefabricated buildings with the use of efficient materials, making it possible to reduce the weight of structural members in comparison with buildings made of traditional materials, to sharply reduce the construction

period, and to protect the territory under development from damage to a maximum degree. In the USSR this trend has become widespread. A series of light buildings using steel, aluminum, and other efficient materials (the weight of which is 20 and more times lower than buildings made of traditional materials) has been developed for construction in permafrost regions. The load on the piers was sharply reduced (by 8-10 times) and this made it possible to solve the problem of foundations - to lower their weight, to change the construction, and to reduce the depth of embedment.

The investigations showed that it is possible to reduce the total loads on the foundations of light buildings up to 35% in calculating the settling caused by the creep of frozen soils, and up to 30% in calculating the bearing capacity of the foundation.

A distinguishing feature of light buildings is the change in the specific weight of temporary loads in the composition of the total loads acting on the foundations. While in traditional buildings they amount to less than 20%, in light buildings they amount to 30-80%. This, in turn, may lead to an increase in the calculated resistances of frozen soils of up to 50%.

COLUMNAR AND PILE FOUNDATIONS ON COMPLEX PERMAFROST SOILS

The use of column and piling foundations (Fig. 2, a, i, k) has found application for buildings with a long period of utilization in one place. Piling foundations, as a rule, are more economical and less demanding of labor.

It is most expedient to use stock metal tubular pilings of small diameter (10-20 cm), sunk into predrilled holes. High-performance light drilling rigs used in engineering and geological surveying may be used for drilling these holes. This significantly reduces the cost of zero cycle operations (it is necessary to bear in mind that in raising buildings made of traditional materials the cost of drilling holes may reach 70-80% of the total cost of building the foundation).

In moving a building to another place tubular metal pilings may be removed from the permafrost with preliminary electric thawing operations. A drawback to this solution is the significant expenditure of metal. Therefore wooden pilings also are used in construction practice; however, their service life is less and they are practically unremovable. Along the top of the pilings the set cap sills, which make it possible to achieve planned levels and layout axes in actual conditions, and heating units are mounted between the cap sills and bases of the frame in order to eliminate cold bridges. For cold buildings (warehouses, barns, and so forth) the necessity for heating units disappears and it is expedient to combine the piling with the bracing of the frame.

In the case of significant horizontal (including wind) loads clusters of pilings are used as is the arrangement of flexible connections between the pilings (Fig. 2k).

An increase in the bearing capacity of foundations of light buildings may be achieved by changing the cross-sectional shape of the pilings (in relation to tubular), and especially (up to 70%) by giving the lateral surface roughness or ribbing. The use of special solutions (limestone, sand-limestone, cement, and so forth) for filling the holes, laying efficient insulation with low heat conductivity in the foundation (in order to reduce the depth of the seasonal thawing of the soils) and also cooling of the soils of the foundation, in particular by using thermopiling, also are practiced. All of these measures in combination with the previously described arrangement of cellars prove to be very effective.

TECHNICAL AND ECONOMIC EFFICIENCY OF DIFFERENT FOUNDATION TYPES IN PERMAFROST CONDITIONS

The technical and economic efficiency of the types of foundation used and their advantages in comparison with the foundations of buildings (made with traditional materials) are obvious from an example, the results of the calculation of which are entered in Table 3.

For stock prefabricated collapsible buildings with a short period of utilization in one place, it is necessary to design foundations with minimum expenditures for earth moving and assembly (disassembly) of their structural members. In most cases it is expedient to construct them according to principle II taking account of the limited development of the thawing cup of the soil. Shallow foundations, using fill or built immediately on the surface of the ground (Fig. 2, b-h), have preference. Wooden sleepers of logs or beams, and also wooden and reinforced concrete pads on which the bracing beams are mounted (or in the case of using pads, the posts of the frame of the building), are widely used. Glued sterilized structural members are resorted to for increasing the service life of wooden foundations.

Column foundations, wooden structures of the cribwork or frame type, and also 3-dimensional metal rod structures (Fig. 2, a-k; 3, a-c) are used where it is necessary to provide ventilation between the building and the surface of the ground. Metal structures are more standardized and transportable, and are highly prefabricated and easily stored upon disassembly; however, the introduction thereof has been held back because of the additional cost of the metal.

CONCLUSIONS

Thus, the development and use of a complex of measures providing for the stability of foundations, in combination with new types and designs of foundations, have made it possible to raise buildings in practically any complex of permafrost conditions. Finding more efficient and economically well founded means of increasing the bearing capacity of weak foundations, made up of permanently frozen saline, highly icy soils and ice, and also further perfection of the structural members of foundations are the next problems for research.

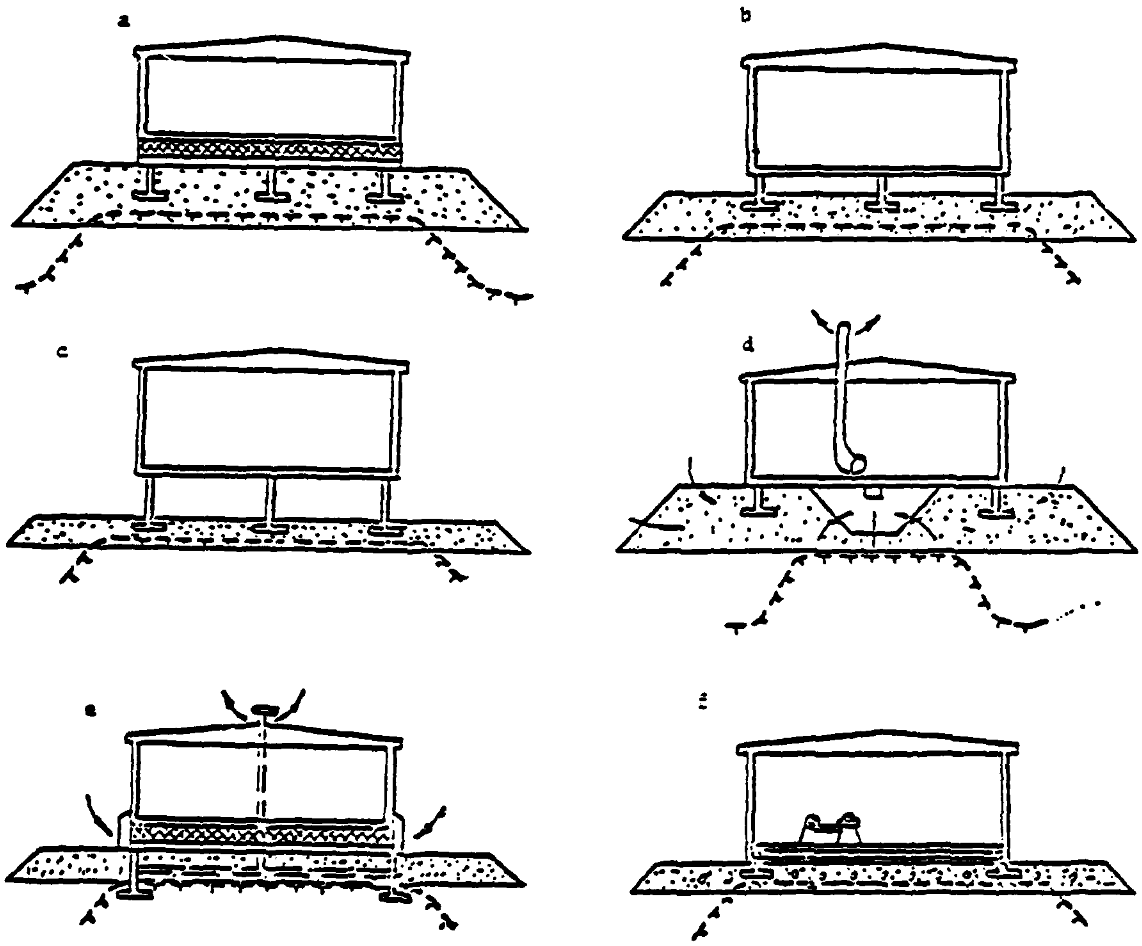

Figure 1. Diagram of the construction of buildings on fill: a - increased insulation inside the floor; b - ventilated cellar; c - unheated first story; d - ventilated fill made of large pore material; e - air cooling ducts; f - brine system of artificial cooling.

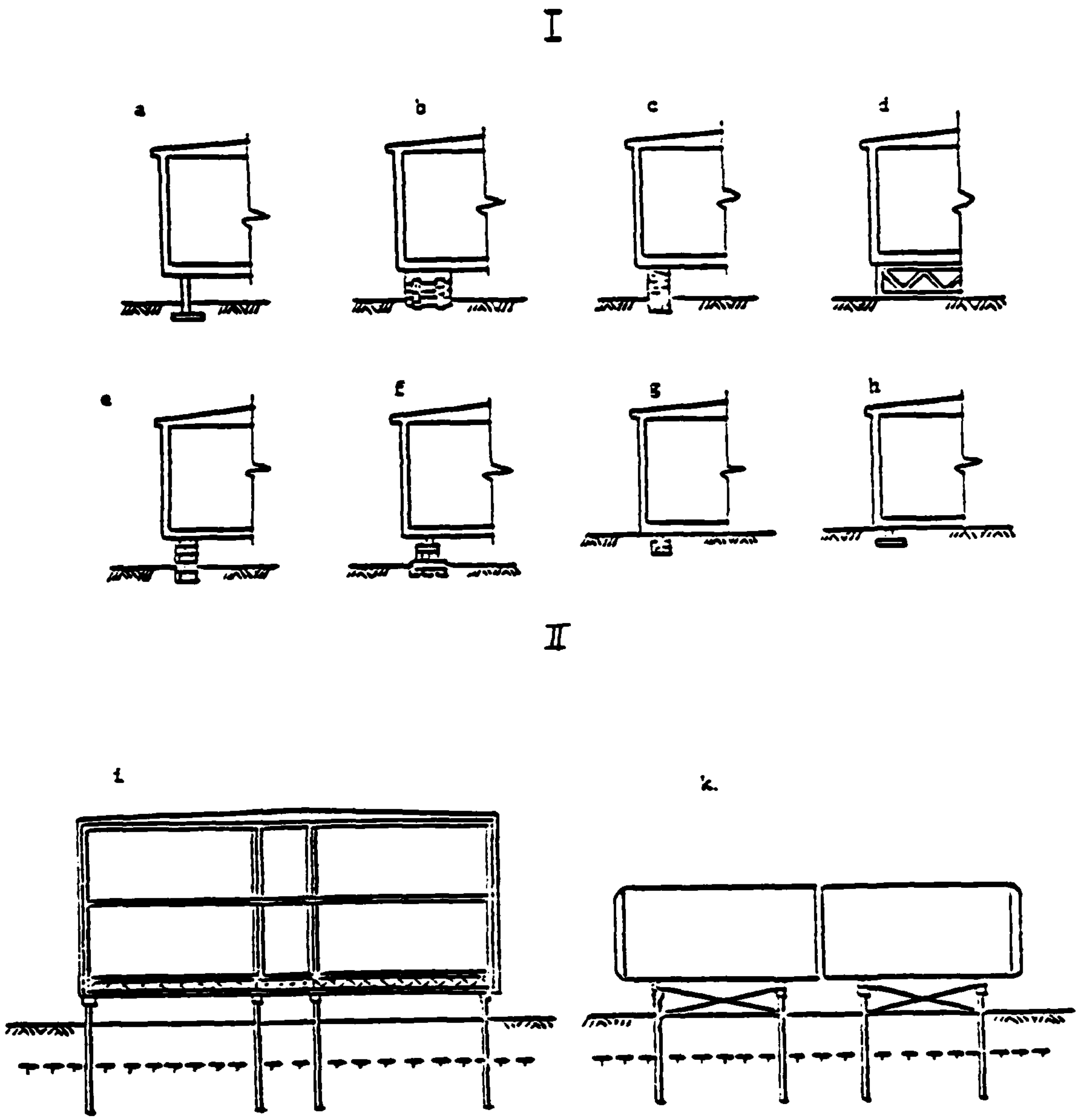

Figure 2. Foundations of light buildings.
I. - shallow foundations;
II. - piling foundations
(a, b, c, d, e, f, i, k - the use of foundation soils according to Principle I; g, h - with the use of foundation soils according to Principle II). a - column reinforced concrete; b - cribwork; c - in the form of cells with ridges; d - frame; e, g - out of rubble or concrete blocks; f - in the form of a wooden pad; h - out of sleepers or columnar; i - piling of metal pipes; k - the same small diameter, connected with flexible braces.

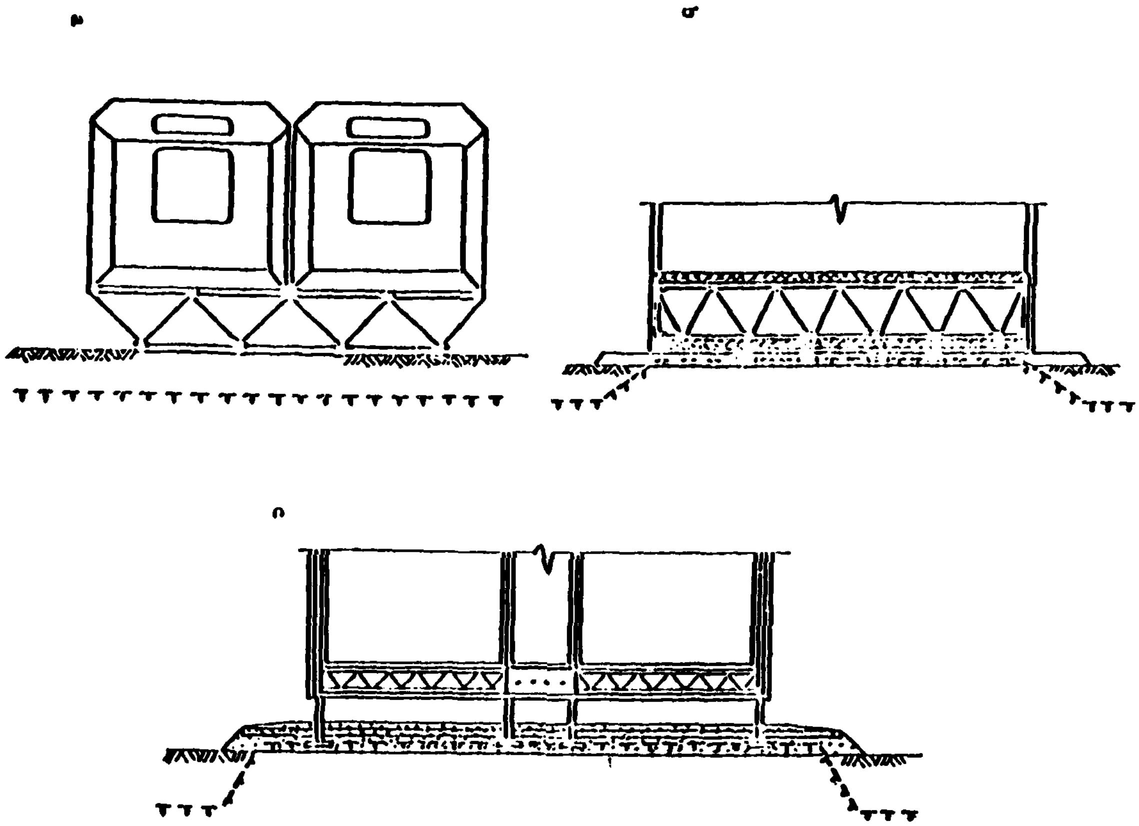

Figure 3. Contemporary types of surface foundations of the LenZNIIEP design: a - foundation of a fully transportable building in the form of a 3-dimensional rod construction; b - foundation of a prefabricated collapsible building in the form of a 3-dimension rod construction, combined with a covering; c - columnar reinforced concrete foundations with efficient heat insulation laid in the foundation.

Table 3.

Economic Indicators of Different Types of Building Foundations in the Territory of the Pechora Coal Basin with an Average Permafrost Temperature of -2°C.

Characteristics of Building	Load on Foundation, kN	Cost and Labor Requirement of 1 Foundation, Rubles / Man Days			
		Column	Piling		
		Reinforced Concrete	r. conc.	metal	wood
Fully prefabricated residential building - hostel for 48 persons with steel frame, with the use of aluminum and efficient materials	160	126 / 7.5	78 / 1.3	60 / 0.9	59 / 1.5
16-apartment residential building with walls of blocks of sand-free concrete and prefabricated reinforced concrete covers.	700	286 / 15.9	191 / 3.6	-	-

CONSTRUCTION OF PILING FOUNDATIONS AND INCREASING THEIR BEARING CAPACITY IN FROZEN SOILS

By A.V. Sadovsky[1] and Y.A. Targulian[2]

INTRODUCTION

In the case of using foundation soils in the frozen state (principle I) piling foundations are the basic type of foundations for buildings and structures.

Piling foundations account for 15-20%, and sometimes even more, of the total expenditures for constructing buildings and structures on permafrost. Of the total estimated cost of a foundation, up to 75% goes for the work in constructing piling foundations, and only a quarter of this amount for the material. The most expensive and laborious part of the work in constructing a piling foundation is performed at the construction site under climatic conditions unfavorable for work in the open air. Of all the expenditures for work in constructing piling foundations in permafrost, 50-80% goes toward sinking holes.

The basic characteristic of the construction and operation of piling foundations on permanently frozen soils is the fact that pilings may only be loaded after they are frozen into the ground, and they may receive the nominal load only after the restoration of the nominal negative temperatures of the currently frozen soils of the foundations. The bearing capacity of pilings in permanently frozen soils depends on their temperature, thus, during the entire service life of a piling foundation the bearing capacity changes. It increases in the case of a drop and decreases with a rise in the temperature of the soils of the foundation.

AN OVERVIEW AND COMPARISON OF PILING TYPES AND INSTALLATION METHODS

The sinking of pilings usually is connected with significant heating of the permanently frozen soils of the foundation, therefore, the freezing of pilings into place (and especially the restoration of the nominal negative temperatures of the soils around the pilings) often lasts for months. Consequently, in accordance with this, the time required for the construction of a building or structure increases.

In keeping with the construction standards, in the case of maintaining foundation soils in the frozen state, pilings are sub-divided into 3 types according to the method of sinking; these are:

a) Drill sinking method: the sinking of pilings into predrilled holes, the diameter of which exceeds (by 5 cm and more) the greatest cross-section of the piling, with the hole filled with a soil solution. Drill

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sinking pilings are used in solid frozen and plastic frozen soils, including those containing large fragment inclusions with an average soil temperature along the length of the piling of -0.5° and lower.

Advantages of the method: applicability in practically any permafrost conditions; the possibility of sinking pilings of any cross-section and length; very little probability of damage to the pilings during the sinking process; accuracy of sinking according to the plan and in depth; the small heating of soils as a result of sinking the pilings; the possibility of cooling the soils of the foundations through the holes prepared for the piling.

Disadvantages of the method: the high cost and labor of drilling holes with a great amount of drilling (exceeding the volume of pilings by 1.6-2 times) and the necessity of preparing and filling the borehole with a soil solution, i.e. the so-called "wet" processes.

b) Sinking method: sinking pilings with heating of the soil. As a rule, heating is performed with steel needles. Sinking prongs are used in solid frozen soils such as fine and powdery sands containing large fragmented inclusions in an amount of not more than 13% and with an average soil temperature along the length of the piling of -1.3° or lower. The diameter of the thawing zone must exceed the greatest dimension of the cross-section of the piling (diagonal) by not more than 2 times.

Advantages of the method are simplicity, accessibility, low cost and labor (explained by the fact that the mechanical destruction of high strength frozen soil is replaced by the thawing thereof), the possibility of sinking piling without the use of expensive drilling techniques, the absence of "wet processes", an increase in the nominal bearing capacity of pilings by 20-30% (in comparison with drill lowered pilings in sandy soils with the cavities filled with a clay solution).

Disadvantages of the method: significant heating of the foundation soils and long term irregular freezing of the piling into place (up to 3-6 months), restoration of nominal negative temperatures of the foundations which increases the construction time, the possibility of the buckling of parts during the process of freezing them into place, the nonapplicability of the method in the case of "high temperature" foundation soils (in the case of an average soil temperature higher than -1.3° in the zone of piling placement) and (in the case of a content of more than 15% of large fragment inclusions, complication of the technique of sinking pilings into sands) additional safety requirements as a result of the possibility of the ejection of the boiling soil mass of a hole.

c) Drilling-hammering method: hammering pilings into predrilled holes, the diameter of which is 1-2 cm less than the smallest cross-section of the piling. Drilling-hammering pilings are used in plastic frozen soils containing less than 5% of large fragments.

Drill casing pilings are a variety of drill hammering pilings. These are hollow round pilings and piling casings with a knife ring sunk by means of drilling a borehole through the cavity of the piling with periodic

driving thereof by hammering. At the present time casing pilings are used in road construction (in building stream crossings). They are little suited for mass, residential, and public construction as a result of the significant complexity of their installation.

In plastic frozen soils without large fragment inclusions, and only during the autumn, when the upper layers of the soil are not in a solid frozen state, the hammering method of sinking piling may be used, that is hammering piling without the preliminary drilling of holes. The possibility of this type of hammering has been determined on the basis of hammering sample pilings.

The advantages of the drill-hammer method are: the insignificant heating of the soils and rapid freezing of piling into place, the accuracy of sinking in the plan, the reduction in the volume of drilling operations by 2-2.5 times (in comparison with the drill sinking method) and the absence of "wet processes".

Disadvantages: the applicability only in plastic frozen soils with a limited content of large fragment inclusions, the formation of a soil plug under the bottom of the piling in the process of sinking these (due to the significant volume of thawed and frozen soil split off or squeezed off the walls by the point of the piling which upon freezing may buckle the piling).

In recent years in regions with high temperature permafrost, the drill hammering method of sinking the piling has been used quite widely. With the development and introduction of the new method of sinking holes, with a steam vibrator, the drill-hammering method of sinking piling has begun to be used even in regions of low temperature (solid frozen) soils.

In using soils according to the first or second principle, when rocky, poorly compressible, large fragmented or sandy soils occur, piling props are used. Piling props are set into drilled holes of large diameter or are hammered into drilled holes of smaller diameter than the diagonal cross-section of the piling, or into thawed holes.

All of the methods of sinking piling used are connected in one way or another with the preparation of the holes. In practice, the rates of performing the zero cycle operations in the case of piling foundations are determined by the rate of sinking the holes or sinking the piling.

PILING INSTALLATION METHODS

The methods of sinking holes in permafrost may be briefly characterized in the following ways.

Impact cable drilling is the only universal method of drilling applicable for any permafrost condition. However, due to low productivity it should be used only in those conditions where other, more efficient, methods are not advised, that is, in the case of a large amount of fragmented material and boulders.

Thermo-mechanical drilling is rarely productive for construction sites with uniform low temperature permafrost and with a limited number of large fragment inclusions, boulders, without significant inter-layers of gravel and pebbles, and without significant ice lenses. In the case of the presence of boulders, large fragment inclusions, and interlayers of gravel and pebbles in the soil, the sinking of holes is greatly slowed down. Widened areas are formed in the boreholes, and the permanently frozen soils of the foundations are heated significantly and irregularly. As a consequence of this, the length of time required for freezing the piling into place is increased significantly, and, consequently, the total construction time also is increased. The presence of widened areas in boreholes also produces an increase in the consumption of a specially prepared soil solution for filling the spaces between pilings and borehole walls.

Rotary drilling of permafrost is productive and inexpensive. In different regions of the Far North commercial drilling cranes are used successfully. The boulders found are opened up by a shaft hammer and removed with a worm conveyor.

Recent years have seen the successful use of universal pile driving rigs, mounted on the base of an escalator, which drill holes 0.3-0.5 m in diameter, up to 16 m deep in permanently frozen soils having large fragmented inclusions. The boulders are broken up by the impact cable method, and the fragments are removed by a worm conveyor. The holes are sunk by a removable drilling drive suspended on the pile driving rig. The worm conveyor of the truck mounted equipment is driven by one or two 60 kilowatt electric motors.

The depth of sinking the boreholes with the truck mounted drilling drive is 8 m if the drive is installed with a diesel hammer and also retained on the drive rails. If the diesel hammer is replaced by a drilling drive it is possible to sink a borehole to a depth of 16 meters. The assembly and disassembly of the truck mounted drilling drive without the removal of the diesel hammer is accomplished in 10-15 minutes, and with disassembly of the diesel hammer in 20-30 minutes. A pile driving rig of the SKV type with one axis of motion can sink a borehole and drive piling under an entire building 12.5 m wide. The assembly and disassembly of the rig is accomplished without auxiliary mechanisms (cranes). Thus, one machine with a crew consisting of two men performs all operations connected with the sinking of holes and hammering of the pilings.

Thawing out holes with steam needles is very productive. The replacement of the process of the mechanical destruction of permafrost by thawing simplifies and lowers the cost of sinking holes by many times. However, the thawing of holes with steam needles involves a significant increase in the time required for freezing the pilings into place.

In recent years, the Scientific Research Institute of Foundations of Gosstroy, USSR, together with the Institutes of Fundament Project, VNIIstroydormash, and the construction in subdivisions of Minneftegazstroy, USSR, had developed a new combined method of sinking piling into permafrost by a steam vibrator. The method combines the basic advantages of the traditional methods of sinking pilings in holes, at the same time being

free of their inherent disadvantages.

The steam vibrator is a tubular structure with an open lower end to which the steam is supplied. A vibrosinker is rigidly attached to the head part of the leader. The steam thaws the frozen soil around the perimeter of the leader and then emerges to the surface through the leader pipe, completely or partially thawing the core of frozen soil entering the leader.

Thanks to the oscillations produced by the vibrosinkers the leader does not "hang" on the walls of the hole but continuously moves behind the thawing boundary. In addition, large or small inclusions are moved aside and broken up and access to the surface of the frozen soil is freed by the steam jet. Since the thawing of the soil precedes the movement of the leader, the cutting edge of the leader does not enter into mechanical reaction with the frozen soil (it does not break it up by mechanical means) but penetrates into the thawed soil. In practice, as a result of the cutting edge not reacting with frozen soil but with thawed soil its service life is increased 10 fold.

The thawing of the soil is limited by the perimeter of the leader. From its outside edge the layer of thawed soil amounts to several centimeters in all. The thawing and liquefication of the soil during the vibration of the leader leads to a multiple reduction of the forces of friction of the leader against the wall of the borehole. The permafrost adjacent to the walls of the borehole is heated insignificantly, and turns into the plastic frozen state at a depth of several centimeters.

The core of frozen soil which enters the leader may be removed from the borehole with the leader.

However, as actual experience in sinking pilings shows, in the case of a short length of pilings, partial melting of the core in the borehole for the purpose of obtaining a soil slurry instead of an extraction of the core is more suitable. The slurry fills the spaces between the walls of the borehole and the piling, and the necessity of a special preparation of a soil solution is eliminated.

With a steam vibroleader it is possible to sink boreholes in sandy and clay soils, irrespective of their temperature, containing up to 40% of large fragment inclusions, and with individual boulders of up to 0.6 times the diameter of the borehole. Boreholes 0.35-0.45 m in diameter are sunk at the rate of 20-40 m/hour. The sinking of pilings into boreholes sunk by a steam vibroleader has features defining the high quality of the boreholes. Thanks to the fact that the walls of the hole are smooth and it is partially filled with a soil slurry, in clay soils it is possible to sink piling into boreholes of large diameter under the action of only their own weight. In sands some more vibration action on the piling is necessary. In addition, such disadvantages of the drill sinking method are high cost and labor in preparing boreholes; however, the necessity of using soil solutions ("wet processes") is eliminated.

The plastic frozen state of the walls of a borehole opens additional possibilities of using the drilling hammering method for sinking the pilings. This method becomes optimum in the case of sinking boreholes with a steam vibroleader both in low temperature and in high temperature permafrost.

As a consequence of the fact that the borehole contains a soil slurry for filling cavities, the diameter of the borehole in the case of the drill-hammering method is not less than, and not more than 5-10 cm more than, the side of the cross-section of a square piling. Upon sinking a piling the slurry is squeezed upwards through the remaining gaps between the walls of the borehole and the piling. This guarantees the sinking of the piling to the bottom of the borehole and the absence of a considerable volume of thawed soil, which would buckle the piling upon freezing, under the face of it. In the case of the above mentioned ratio of dimensions of the cross-section of the piling and borehole, the drilling volume is reduced by 30-40% in comparison with the drill sinking method. The piling is hammered evenly since only the corners of the piling penetrate the plastic frozen walls of the borehole. For sinking drill-hammered piling into clay soils, both impact and vibration action on the piling are acceptable, and only vibration action is acceptable in sandy soil. The high productivity of sinking boreholes with a steam vibroleader introduces changes into the organization of the zero-cycle operations. The simultaneous use of equipment with a steam vibroleader for sinking boreholes and a pile driver for handling piling, which in combined operation sink 30-40 pilings into permafrost to a depth of 6 m in a shift, becomes efficient.

The use of a steam vibroleader makes it possible to extend the drill-hammering method for sinking piling to the entire territory with permafrost. In addition, while retaining the well known advantages (rapid freezing of the piling into place, and so forth) the drill-hammering method is freed from its inherent drawbacks.

The new method of sinking pilings also has similarity with the method of sinking piling into boreholes thawed out with a steam meter. The similarity exists in that with both cases the mechanical working of high strength frozen soil is replaced by thawing it out with steam (which is basic for a significant reduction in the cost of sinking boreholes). However, the volume of the thawed soil in the case of sinking boreholes under drill-hammered piling with a steam vibroleader is 4-8 times less than in the case of sinking boreholes with a steam needle, therefore, the energy costs are incomparably less, and the heating of the foundation soils is also less and more uniform.

The small amount of heating of the foundation also is promoted by the following: the high rate of sinking, the removal of part of the soil core from the borehole, the leaving of pieces of frozen soil in the borehole (which, upon thawing in the hot slurry, rapidly cool it) and the squeezing of part of the thawed soil to the surface during the sinking of the piling. As a result, in the case of sinking boreholes with a steaming vibroleader, the soils of foundations are heated less than in the case of impact cable drilling. Correspondingly, piling sunk with holes drilled by a steam vibroleader is frozen into place more rapidly and uniformly, and the

nominal negative temperatures of the foundation soils around them are disturbed more rapidly.

The sinking of boreholes with a steam vibroleader (causing the transition of soils on the walls of the boreholes from the solid frozen state into the plastic frozen state) also is creating grounds for extending the area of application of screw pilings.

With a steam vibrator it is possible to sink boreholes with almost any base machine: pipe layers, excavators, cranes, and so forth. Therefore, the application of this method makes it possible to reduce sharply the cost and increase the productivity of drilling operations without substantial expenditures for acquiring expensive drilling equipment.

At the present time, different measures (making it possible to reduce the cost and labor) of constructing piling foundations in permafrost with the simultaneous reduction in the time of the zero-cycle operations and the overall duration of foundation construction are being developed. These measures basically involve the perfection of the technique of sinking pilings and, primarily, the technique of sinking boreholes, improving the construction properties of frozen soils and foundations by reducing their temperature, and improving the construction of pilings.

Compared with the technique of sinking pilings used at the present time, the complex structure of expenditures, the increase in productivity, and the reduction in the cost of operations for sinking boreholes, this means of reducing the cost in the construction of piling foundations is most efficient.

The development and broad application of sinking methods is a fairly simple measure for reducing the cost and increasing the productivity of piling operations: a) the rotary method with the destruction of lag boulders by impact action and b) the steam vibroleader.

In the case of the drill sinking method of sinking piling, the reduction in the dimensions of the pilings may be achieved with the use of a stronger filler in the gaps between the walls of the borehole and the piling.

At the present time, a clay solution is poured into the borehole. In the case of constructing a foundation in sandy soils the piling seemingly is in an envelope of a weaker material than the foundation soil. In this case, the use of another filler, even if equal in strength to the permanently frozen soils of the foundation (for example, sand) instead of the clay solution (in accordance with the standards), makes it possible to increase the normal bearing capacity of piling or in the case of the same bearing capacity to reduce the dimensions of the piling by 20-25%.

The surface of freezing of the cavities (with the filler soil solution) and the walls of the boreholes is 15-25% greater than with the surface of the piling. In addition, the shear resistance of the soil, as a rule, is 10-20% higher than along the lateral surface of a reinforced concrete or wooden piling. Therefore, the increase in the strength of freezing the

filler of the cavities with the surface of the piling of up to 50% in clay soils, and up to 40% in sandy soils, in comparison with the strength of freezing the piling with a clay solution is almost equivalent to the same increase in the bearing capacity of the piling. In addition, the shear resistance of the soil solution along the surface of the piling and along the perimeter of the borehole becomes identical. A further increase in the strength of freezing the piling with the filler of the gaps is useless, since the resistance of the soil along the perimeter of the borehole is the same.

EFFECTS OF TEMPERATURE REDUCTIONS

An improvement in the structural properties of the frozen soils of the foundations by reducing their temperature produces a great effect.

The bearing capacity of hanging pilings in permafrost depends on the temperature thereof. The bearing capacity of pilings increases markedly in the case of a reduction of the temperature of the permafrost and the transformation thereof into the solid frozen state. Reducing the nominal temperature of plastic frozen soil by from 1-1.5°C increases the bearing capacity of pilings by more than two times.

In most of the regions of the Far North the temperatures of the permafrost of foundations, used according to the first principle, are reduced after constructing buildings with ventilated cellars. The construction properties of the soils of foundations are improved and the bearing capacity of piling foundations increases. Reserve levels of the bearing capacity are manifested even after the first winter season of operation of a ventilated cellar and they increase with the course of time. However, they are in no way used since in the case of planning piling foundations the dimensions thereof are determined on the basis of the strength properties of frozen soils under natural temperatures. A reduction in the temperature of plastic frozen soils of foundations, performed ahead of time or in the process of construction and taken into account in planning, makes it possible to reduce the dimensions or number of pilings in a foundation by up to 50%.

Methods of cooling permanently frozen soils of bases are divided into three groups: cooling from the surface, through boreholes, and with cooling equipment.

Cooling from the surface is most effective in regions with a great snow cover. It is performed by means of periodically removing the snow from the ground during the wintertime, by means of the preliminary construction of a ventilated cellar, or a system of pipes low in the seasonal thawing layer of the soil.

In southern low snow regions of permafrost, shading and even insulation of the surface of the ground during the summertime promote a reduction in the temperature of permanently frozen soils.

Cooling soils through boreholes drilled for sinking piling is a more rapid method, requiring less changes in the planning of construction

operations. The frozen soils of foundations are very simply cooled during the wintertime by the cold outside air. For several weeks the soils of the foundations may be cooled through the borehole until their temperature corresponds to the temperature which is established after several winter seasons of utilization of a building with a ventilated cellar.

Cooling of foundation soils through hollow pilings is similar to cooling through boreholes, and is even more effective. With hollow pilings it is possible to cool the foundation soils with cold air ventilation in the case of sinking not only drill sunk piling, but also sinking and drill-hammered piling.

An improvement in the construction properties of frozen soils by cooling does not require great expense. Efficient planning, organization, and working techniques are fundamental. We obtain a sort of substitute for the operation of expensive drills and also cranes and transportation; in the operation of ventilators with air cooling through boreholes in periodic bulldozer excursions to the construction site, in the case of cooling from the surface.

Cooling the soils of foundations with self regulating liquid and steam-liquid arrangements is used primarily where one time cooling from the surface or through boreholes does not give the necessary result.

METHODS OF IMPROVING PILING CONSTRUCTION

One of the simple methods of improving the construction of pilings at the present time is the use of combined wood-reinforced concrete and wood-metal pilings.

The lower part of the piling during the course of the entire service life of a building or structure is located in permafrost. This part of the piling may be wood. In permafrost wood is not subject to rot. The nominal shear resistances of permafrost to concrete and wood are identical, and the strength of wood in the direction along the fibers is on the same order of magnitude as the strength of concrete. Therefore, the use of composition wood-reinforced concrete pilings is not connected with an increase in their dimensions. In the case of the frozen hanging piling 8-16 m long used at the present time, replacement of the lower part of the piling by wood would make it possible to reduce the consumption of reinforced concrete by almost two times.

The use of wood-metal pilings instead of metal piling is even more effective. The nominal strength of the freezing of smooth metal pilings with the soils of foundations is 30% less than the strength of freezing together with wood. Therefore, replacement of the lower part of metal pilings with wood makes it possible to reduce the depth of sinking pilings while retaining the nominal bearing capacity. In this case, the cost of a piling foundation is reduced not only as a result of the reduction of the cost of the material of the piling itself, but also as a result of the reduction of drilling, the amount of the solution poured into the borehole, and so forth.

In the case of construction in permafrost, the time required for the construction of composite piling, and the achievement of the calculated bearing capacity by them, often significantly exceeds the time required for the assembly of the above ground part of a building or structure. Reduction in the time required for zero-cycle operations and, in particular, the time required for freezing piling into place and restoration of the nominal temperatures of the foundation soils can reduce the general construction time very significantly, and produce an additional economic benefit from the early introduction of a building structure into operation.

The time required for freezing piling into place depends on the method of their sinking, and even on the type of drilling and pile driving machines used.

Pilings take longest to freeze into place when they are sunk into boreholes thawed out by steam needles. The replacement of this method of sinking boreholes by the drill sinking method or drill-hammering method can immediately reduce the time required for freezing piling into place and restoring the nominal temperatures of foundation soils from several months to several weeks and even days. Correspondingly, the total time required for the construction of a building or structure is reduced by several months.

In the case of the replacement of impact-cable drilling by rotary drilling or steam vibroleader sinking, the time required for sinking drill-lowering pilings may be reduced by 2-3 weeks.

In the case of the use of drill sinking piling usually a sand-clay solution with a moisture content of 0.3-0.5, which only turns into the solid frozen state upon cooling to -0.5 to -1.0°C, usually is used. The moisture content of the sand filler of the gaps is half that and at a temperature of -0.1 to -0.3°C it is already frozen solid. In the case of the use of piling foundations in high temperature permafrost soils, the replacement of the sand-clay filler of gaps by a sand filler may accelerate the time required for freeing pilings into place from 1-2 months.

The sinking of pilings into boreholes with vibration makes it possible to reduce the ratio of the dimensions of boreholes and pilings, and to reduce the volume of the soil solution poured into the borehole by half, and correspondingly, to reduce the time required for the freezing of pilings into place.

A very efficient means for reducing the time required for freezing pilings into place is the cooling of foundation soils.

In the case of the cooling of soils on the surface by means of removing snow from the area, in regions with high temperature permafrost and with great snow covers, the time required for freezing drill sinking pilings into place is reduced by 1-1.5 times, and the time required for sinking pilings sunk with melting of the soil by steam needles is reduced by 2-3 times. In the case of cooling soils through boreholes or hollow pilings with cold air, the time required for freezing pilings into place is reduced by 10-20 times. The time required for freezing pilings into place

and restoration of the nominal temperatures of foundation soils may be significantly reduced also by the use of liquid and steam-liquid cooling arrangements.

CONCLUSIONS

Thus, the basis of reducing the expenditures for constructing composite pilings, both with respect to cost and with respect to labor and time, involves the operations of the zero-cycle. With the use of progressive methods of sinking pilings, and, first of all, with the method of sinking boreholes, perfection of the technology and organization of operations in constructing the boreholes of foundations make it possible to reduce both the cost of polyfoundations and the expenditure of time and labor for their construction in permafrost by up to 2-3 times.

IMPROVEMENT OF BEARING CAPACITY OF PIPE PILES BY CORRUGATIONS

By H.P. Thomas[1] and U. Luscher[2]

ABSTRACT

About half of the trans-Alaska oil pipeline is supported above ground on nearly 80,000 special piles called vertical support members. Design and construction of these piles called for innovative solutions. This paper deals with one of the innovations developed during the design of these piles. The smooth surface of pipe piles can be roughened in situ by means of a modified well casing expander tool called a pile corrugator. Tests and studies carried out on the trans-Alaska pipeline project showed a resultant improvement in pile bearing capacity in unfrozen as well as frozen soil profiles. The paper describes the potential benefits of corrugating pipe piles for foundations together with the limitations of this new technique.

INTRODUCTION

Adfreeze bond generally controls the design of smooth piles in permafrost. Numerous published load tests (Crory, 1963) have shown that adfreeze bond failure is sudden and abrupt and strength can only be partially regained after motion stops. One of many innovative aspects introduced on the trans-Alaska pipeline was use of an oil industry tool to corrugate steel pipe piles to support elevated portions of the pipeline. This paper will briefly describe the corrugator, the benefits it provides and the limitations of its use.

CORRUGATOR

Depicted in Fig. 1, the corrugator is an oil industry tool (manufactured by Lynes Co., Houston) which has been used to expand casings in oil wells. It consists of a set of radial forming dies activated by a steel-reinforced rubber bladder. The bladder is expanded by hydraulic fluid subjected to a pressure on the order of 3500 psi (246 kg/cm^2). The corrugator unit is lowered to the bottom of the pile and then is activated and raised successively. To produce a series of corrugations at 12-in (30-cm) spacing, the double corrugator with dies 24-in. (61-cm) apart was raised first 1 ft (30 cm), then 3 ft (91 cm), then 1 ft (30 cm) and so forth. Although the pile may be filled with water, this does not affect the operation of the corrugator.

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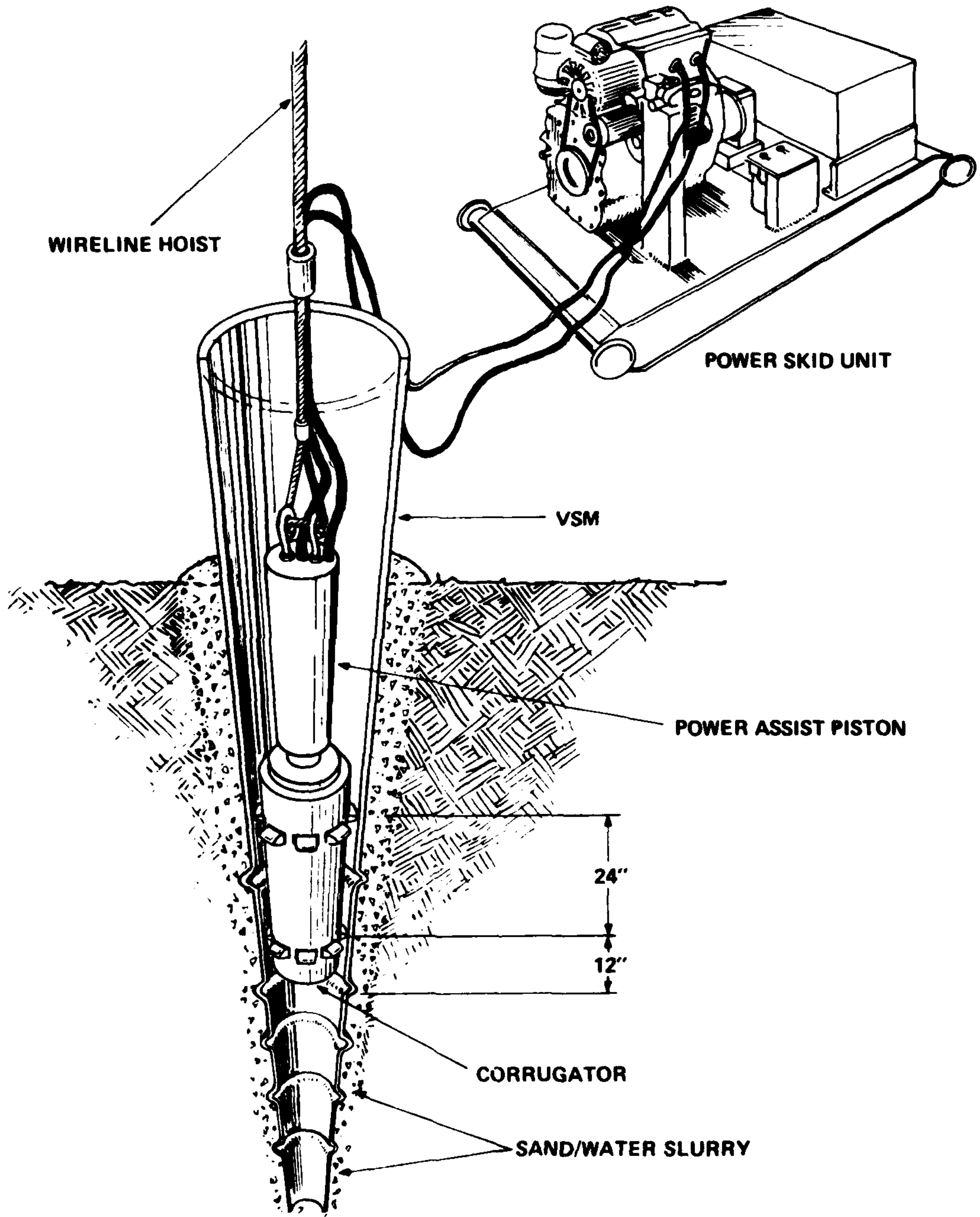

Figure 1. Corrugation system.

Outside diameter of the Alyeska pile was 18 in. (46 cm), wall thicknesses were 3/8 and 1/2 in. (1 cm and 1.3 cm), and depth of the corrugation formed was nominally 3/4 in. (2 cm). Corrugations were made only in the bearing layer; corrugation above this tends to increase frost jacking or downdrag forces and decrease the pile lateral load capacity. Tests in air showed that the pile shortened by about 1/4 in. (6 mm) per corrugation and that the pile axial stiffness was reduced by about one half.

BENEFITS

Although direct driving of piles in permafrost is generally not feasible, corrugations may be used with directly-driven closed-ended pipe piles, with piles driven into undersize predrilled holes, and with piles slurried back in oversize predrilled holes. In the latter case, the piles may be precorrugated, but the best mechanical interlock is obtained by corrugating after the pile has been installed and the annulus has been filled with slurry.

For corrugation in-situ, improvement in pile capacity is significantly greater than the increase in pile circumference alone would justify. An early field load test on 18-in. (46-cm) pipe piles driven into 16-in. (41-cm) undersized predrilled holes showed an approximate quadrupling of long-term yield strength at a frozen gravel site, albeit a lesser benefit at an ice-rich silt site where soil arching presumably played a lesser role. Alyeska's final design was for 24-in. (61-cm) oversized predrilled holes with the 3-in. (8-cm) annulus filled with a sand/water slurry which was densified by vibrating the pile. Finite element calculations (see Fig. 2) showed that the full slurry shear strength could be mobilized by 3/4 in. (2-cm) corrugations. This enabled the designers to take advantage of the shear strength of the controlled slurry and avoid the "weak link" that smooth adfreeze bond represented.

Probably the best information on the benefit of corrugations is the extensive series of laboratory tests made for Alyeska at the University of Illinois in 1975 on 18-in. (46-cm) diam piles. The length of pile embedded in sand was 11.3 ft (3.4 m) and results of load tests for frozen and thawed condtions are presented in Figs. 3 and 4, respectively. Although these piles had been precorrugated, the difference in behavior between the corrugated and smooth piles is striking. In both figures, there was no "brittle" failure with the corrugated pile. Instead, the corrugated pile continued to pick up additional load-carrying capacity with additional displacement, very similar to the prediction of Fig. 2.

LIMITATIONS

About 50,000 piles were corrugated on the Alyeska project. The greatest drawback encountered with the technique during pipeline construction was that certain makes and batches of pipe steel had a tendency

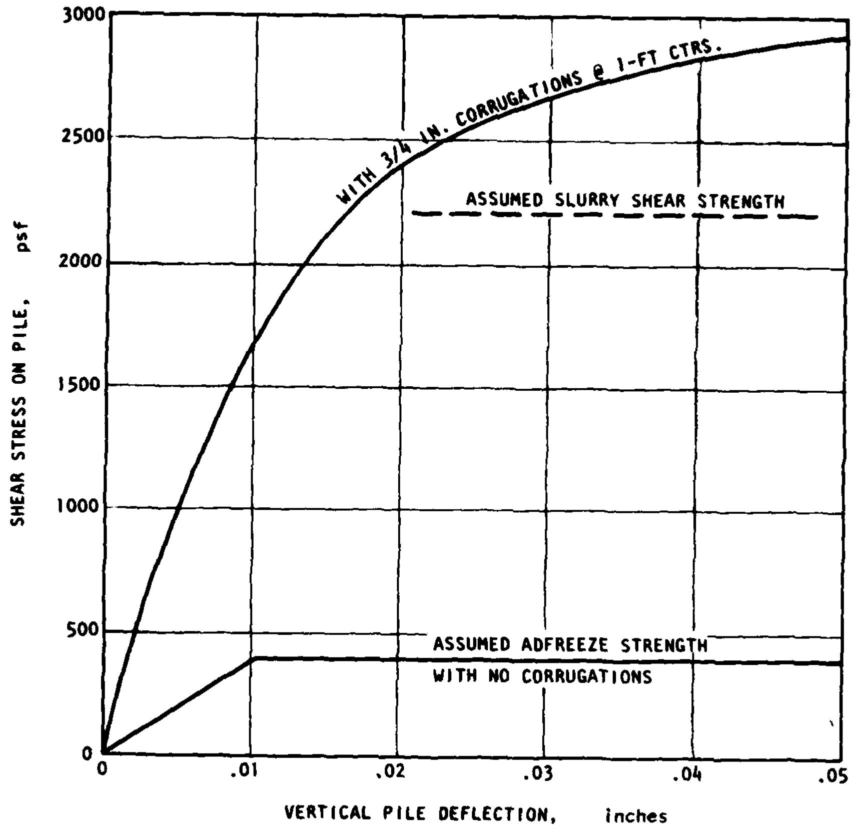

Figure 2. Finite element results.

to split when corrugated. Modification of the corrugator to ensure a uniform upset not exceeding 3/4 in. (2 cm) seemed to help eliminate this occurrence and, as much as possible, steel which had a splitting tendency was utilized for piles not requiring corrugations. It is important that welds in electric arc welded pipe be fully normalized and that ductility requirements (8.3 percent circumferential strain in the case of the Alyeska piles) be clearly specified when ordering pipe. This may, of course, add an increment of cost to the steel.

Another limitation was that very stiff (e.g., frozen) soils tended to restrain the corrugator such that the full depth of 3/4 in. (2 cm) was not always obtained. For this reason, it is desirable that corrugating be done soon after pile installation, before the slurry freezes.

CONCLUSIONS

Use of corrugations significantly improves the bearing capacity of pipe piles in frozen or unfrozen granular soils. This new technique

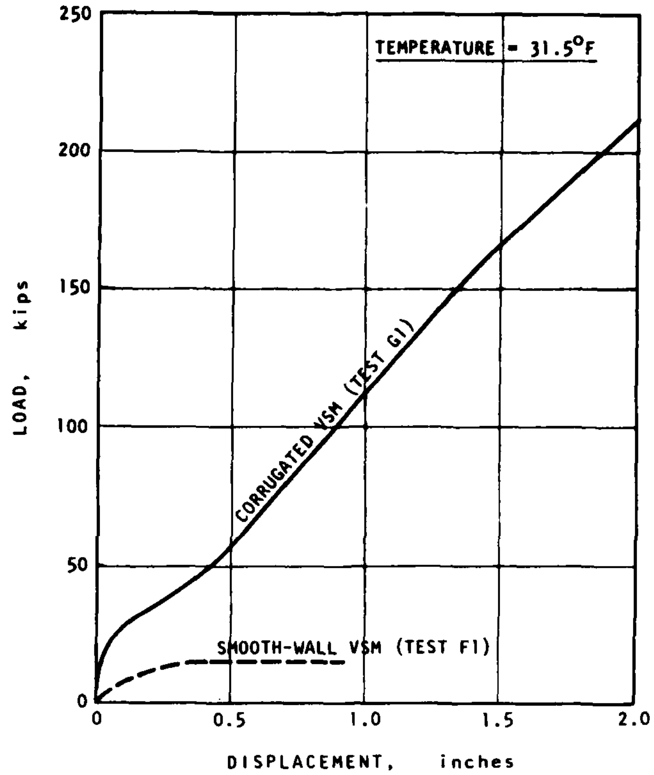

Figure 3. Effect of corrugations on VSM in frozen soil, University of Illinois VSM tests.

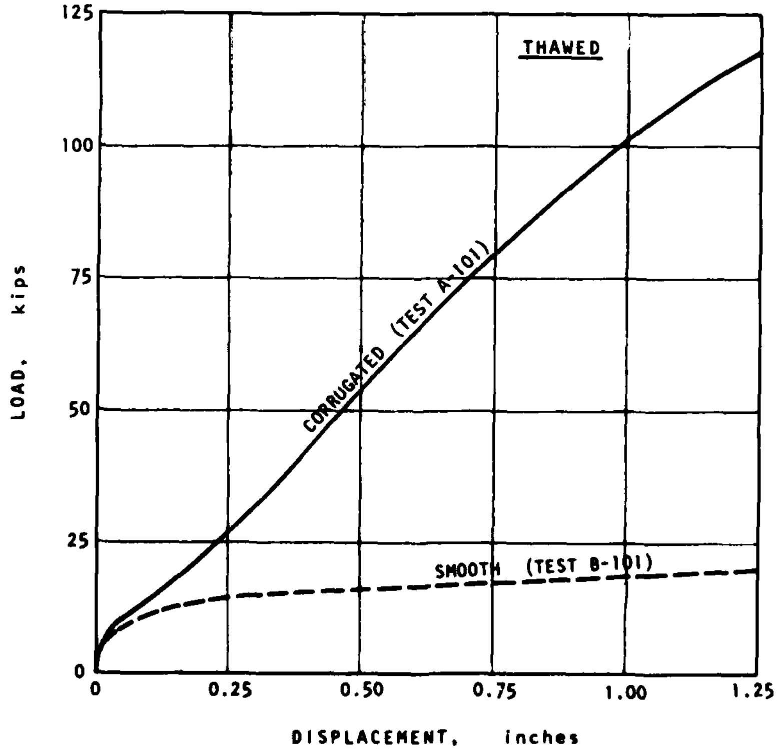

Figure 4. Effect of corrugations on VSM in thawed soil, University of Illinois tests.

was successfully used on the trans-Alaska pipeline project and appears to have applicability in temperate as well as arctic regions. Load tests should be made to evaluate the effect of corrugations on a given pile at a given site.

ACKNOWLEDGEMENTS

Acknowledgement is made to Alyeska Pipeline Service Company for permission to publish this paper, to Raymond International, Inc., who pioneered the corrugation concept, and to the University of Illinois who conducted the cited tests for Alyeska.

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SECTION III:
THE ASPECTS OF GEOTHERMAL FOUNDATION STABILIZATION ON PERMAFROST SOIL CONDITIONS

PERFORMANCE OF A NATURAL CONVECTION HEAT EXCHANGE SYSTEM FOR SUBGRADE COOLING OF PERMAFROST

By J.L. Barthelemy[1]

ABSTRACT

During the summer of 1976, personnel from the Civil Engineering Laboratory erected a buidling on the permafrost near Barrow, Alaska. The structure, placed on just .3 meters of gravel, had been used as a test bed to evaluate an experimental subgrade cooling system. The cooling system consists of 15 loop-configured heat exchangers called convection cells. During the winter months, heat losses from the building into the permafrost are redirected via these convection cells to the cold ambient environment outside, thus preventing progressive degradation from thaw. To date, data collected from thermocouples located in the subgrade and heat-exchange systems have shown that the rate of winter heat removal is even greater than originally predicted.

NOMENCLATURE

d_i = inside diameter of ice shell around a heat-intake pipe

D = ratio of outside-to-inside diameter of ice shell around a heat-intake pipe

k = thermal conductivity of ice

ℓ = length of heat-intake pipe

L = latent heat of fusion of ice

$\dot{Q}$ = rate of heat removal by a single convection cell

$\dot{Q}_t$ = rate of heat removal by the subgrade cooling system

R = overall thermal resistance of a convection cell

t = time

T_a = ambient air temperature

T_p = surface temperature of heat-intake pipe

θ = ratio of (heat-intake) pipe-to-air temperature, $\frac{T_p}{T_a}$

ρ = density of ice

τ = dummy variable of integration

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INTRODUCTION

At the present time, construction of heated buildings on ice-rich or frost-susceptible frozen ground relies, most usually, on the use of thick pads of gravel or other stable soils as a seasonal heat sink to preserve the frozen state of the underlying natural ground, and thus ensure the integrity of the structure. Although lightly loaded structures are frequently elevated above the surface of the pad to provide an insulating air space, this is quite difficult to do with heavily loaded structures such as garages or aircraft hangars, which are instead usually constructed on thicker gravel pads. Well designed combinations of structure and pad have proven highly reliable for construction on frozen ground. However, a problem exists which is becoming more apparent as development of the North continues; that is, scarcity of gravel in many areas, especially in the northern coastal plain of Alaska west of the Colville River.(1) For coastal development, gravel has traditionally been mined from beaches, but in the vicinity of Barrow, for instance, this activity has been considered to have caused accelerated beach erosion and shoreline regression, and is currently not permitted.(2) Inland from the coast, of course, transportation costs and logistics can make gravel a scarce commodity.

Gravel requirements for heated structures can be reduced by incorporating various insulation materials into the pad; however, to seriously reduce this requirement, it is necessary to provide some means of extracting heat from the ground below the structure. At the Second International Symposium on Cold Regions Engineering, held at the University of Alaska in August 1976, Mr. J.E. Cronin[2], from the Civil Engineering Laboratory, presented a paper (3) outlining construction plans for a building on permafrost which would use an experimental system of natural convection heat exchangers for subgrade cooling. During the two months following that conference, a three-man CEL field team, consisting of a mechanical engineer, geologist and equipment specialist, erected the building and cooling system at a site in the Naval Arctic Research Laboratory (NARL), compound near Barrow, Alasaka. A number of thermocouples were placed on the heat exchangers and in the ground so that temperature data could be collected on a weekly basis throughout the year. In addition, a stable benchmark was placed near the building, and level measurements have been taken along the foundation and floor at the end of each summer and winter to detect seasonal heaving and settling.

The test structure, a nominal 16.5-meter by 12.5-meter ATCO "fold-a-way" building, was originally purchased for equipment cold storage and was therefore uninsulated. As a result, heat input into the ground is provided and controlled by a grid network of thermostatically controlled electric heat mats buried beneath the floor. The mats are divided into eight circuits on separate thermostats to better maintain the desired temperature conditions uniformly across the floor area. The subgrade cooling system consists of 15 self-powered heat exchangers which will hereafter be referred to as convection cells. Each convection cell is

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built as a loop, and consists of two connecting half-loop heat exchange surfaces. The heat-intake half-loop is placed horizontally in the ground beneath the floor and joins the heat rejection half-loop, which rises vertically outside the structure. Thus, so long as the ambient temperature is lower than the ground temperature beneath the building, there is a natural circulation of the liquid refrigerant around the convection cell. In this way, heat losses from the building into the permafrost are redirected via the heat exchangers to the cold ambient environment outside.

This paper describes the development and testing of the convection cell heat exchangers, and erection of the building and subgrade cooling system at NARL. Also, performance characteristics monitored from thermocouple and level data during the nearly two years of operation are presented.

BACKGROUND

The concept of the self-regulated heat exchanger is not a new theme in polar regions. Two types of passive, one-way refrigeration devices have been used for nearly a decade in Alaska and Canada to stabilize piling in permafrost.(4) Externally, the two are similar in appearance; however, internally one type operates using a single-phase liquid-convection heat transfer while the other uses a two-phase boiling-liquid and vapor convection heat transfer. Historically, the liquid convection and two-phase devices have been known in Alaska and Canada as the Balch and Long Thermopile respectively, although more recently a number of variations such as freezing cell (liquid convection), THERMO-TUBE[R] (liquid convection), thermosiphon (two-phase) and Cryo-Anchor[R] (two-phase) have been introduced. The Cryo-Anchor, developed by McDonnell Douglas Corporation to refrigerate the vertical support members on elevated sections of the Trans-Alaska Pipeline, is the topic of another report presented at this seminar.

In 1969, the Civil Engineering Laboratory began to investigate the potential of using passive refrigeration devices to accelerate the natural growth rate at the underside of an advancing ice sheet. Initially both liquid-convection and two-phase cells were evaluated in cold chamber tests. However, it was early decided to develop only the single-phase liquid-convection model, which was subsequently named the freezing cell. (5, 6) The Navy demanded a simple and compact heat exchanger that could be flown as conventional piping components to remote locations, and then assembled, placed, and charged on site by two or three men. Thus operational requirements, rather than the relative thermal efficiency, or heat transfer potential, spearheaded development of the liquid-convection concept[3].

Investigations related to the subsurface thickening of sea ice were completed in 1973, by which time the gravel moratorium at Barrow

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(3) Limited cold-chamber tests conducted in seawater at the freezing point temperature indicated that for equivalent heat exchange area, the liquid-convection cell would produce more ice than the two-phase cell at low ambient air temperatures, whereas the opposite was true as air temperatures approach the freezing point of seawater.

was already in effect. Previously, CEL had purchased an uninsulated building for equipment cold storage at the Naval Arctic Research Laboratory. It was decided in early 1975 to use the freezing cells and building as a test bed for a subgrade cooling system.

CONVECTION CELL DEVELOPMENT

The liquid-convection freezing cell used for subsurface ice growth was basically just a closed-ended pipe positioned vertically in an ice sheet so that part was exposed to the cold air above and part to the warmer seawater below, as illustrated in Figure 1. The upper heat-

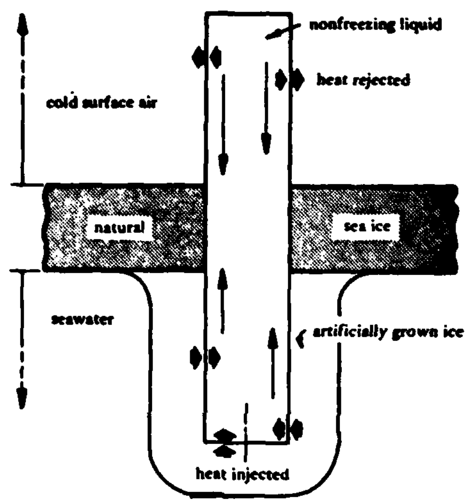

Figure 1. Liquid-convection ice-thickening device.

rejection surface, called the cooling head, was fitted with fins to increase heat transfer. Liquid density differences generated within the cooling head and lower heat-intake pipe resulted in natural convection circulation. For modification to a subgrade cooling system, it was at first conceived to slant the heat-intake pipes to a near horizontal position under a building constructed at grade. However, cold chamber tests of slanted cells placed in freshwater cooled to 0°C resulted in the growth of a severely tapered ice shell.(3) It was obvious that the near-horizontal position of the heat-intake pipe caused a resistance to natural convection forces that virtually eliminated all circulation toward the bottom of the pipe.

Next it was decided to join two 90° slant cells top and bottom. It was thought that a loop-configured convection cell, where flow within the pipes is unidirectional rather than concentric, would result in a more uniform flow pattern and thereby avoid the problem of tapered ice growth. Three generations of loop-configured convection cells followed,

culminating in the final design.

The first generation loop was constructed using cooling heads claimed from old freezing cells, as shown in Figure 2. However, since

Figure 2. Cold chamber test of reclaimed Barrow cooling heads in balanced loop configuration.

there were not enough reclaimed to build an entire subgrade cooling system, additional cooling heads with increased fin area were manufactured. Loops made from these improved cooling heads were evaluated during the second generation tests.

Merely joining two 90° slant cells produced a balanced loop configuration. Thus as the heated refrigerant left the heat-intake pipe and started to flow upwards, the cool-down action of the first cooling head actually served to counteract and slow down the flow rate. It wasn't until the liquid started down the second cooling head that circulation was reinforced by heat rejection. The third generation tests again used

the improved cooling heads, but this time connected in parallel above one arm of the heat-intake half-loop: two 10-cm diameter finned pipes were fed by a smaller unfinned 5-cm pipe rising from the other arm of the heat-intake half-loop. Figure 3 shows the parallel heat exchanger

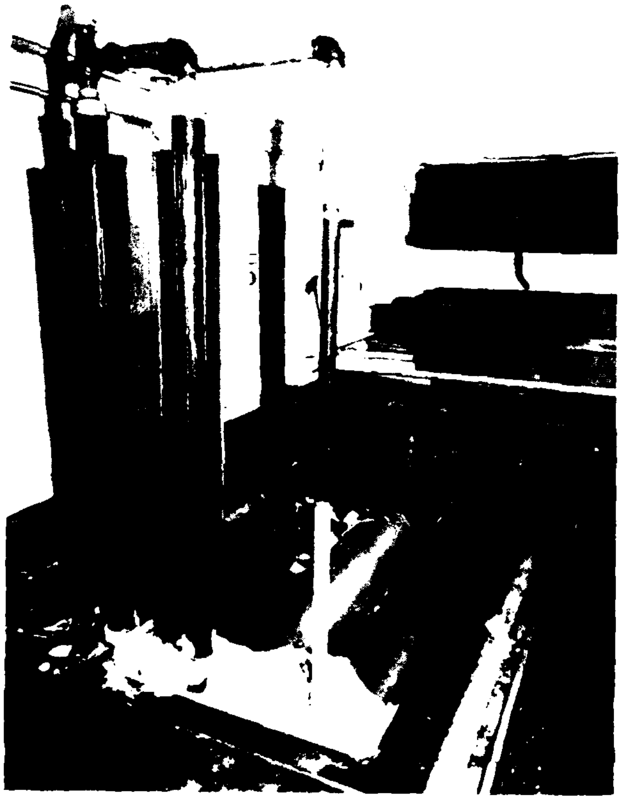

Figure 3. Cold chamber test of improved cooling heads in parallel loop configuration.

and the nearly uniform ice growth that resulted. Figure 4 is a graphical comparison showing the average working temperature (measured at points along the outside surface of the heat-intake pipe) history of the (balanced) improved cooling head and the parallel (improved) cooling head configurations. Not only did the latter operate at consistently colder temperatures, but the liquid flow rates, which were measured visually through a clear plastic section of 5-cm diameter pipe, doubled, reaching a maximum value of just under 10-cm per second. (3)

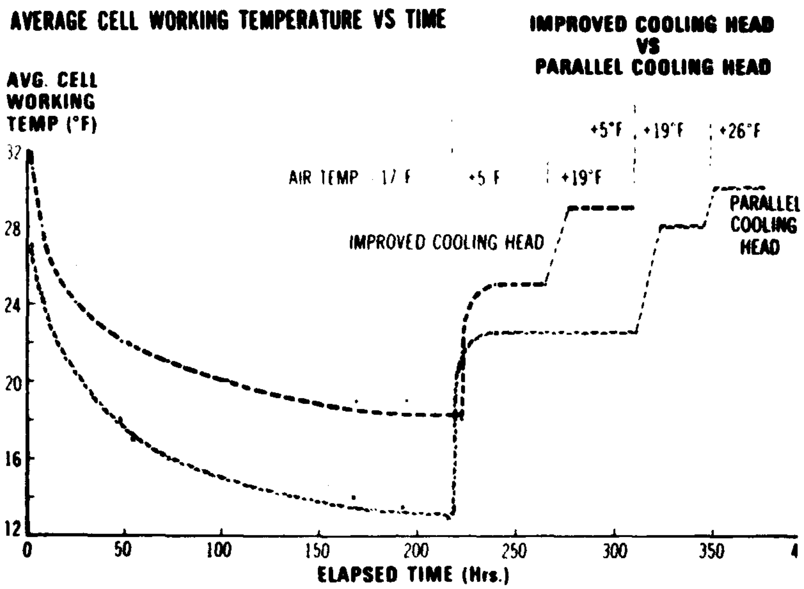

Figure 4. Thermal comparison of (balanced) improved cooling head vs. parallel (improved) cooling head.

Thermal Performance

In order to design the subgrade cooling system and predict future performance, it was necessary to determine, quantitatively, the heat transfer characteristics of the convection cell. In the field installation, the rate at which heat is removed from the soil is related to the temperature of the buried heat-intake pipes. Since the convection cell heat exchanger is a thermal resistance device, the temperature of the heat-intake pipe is a function of air temperature and the total resistance to heat flow through the soil, through the convection cell, and from the cooling head to the surrounding air. In order to determine the overall heat-exchanger resistance (that is, combined liquid natural convection resistance within the piping and forced-air resistance from the cooling heat), the convection cell was tested as a freezing cell in 0°C fresh water.

All cold chamber tests were conducted at a continuous average air temperature and speed of -27°C (-17°F) and 1 m/sec, respectively. In this way, near steady-state conditions existed at each instant in time since heat capacity effects due to temperature adjustments within the growing ice mass and liquid refrigerant were very small compared to the

overall heat transfer rate. In the steady-state approximation, the time-changing temperature of the heat-intake pipe is always some fraction of the ambient air temperature. Also, since water at the freezing point temperature produces concentric rings of ice around a cold pipe, the thermal resistance of the slowly growing ice shell is readily represented by basic heat transfer considerations. Thus from Reference (7), one has the following relationship between heat-intake pipe temperature, air temperature, overall convection cell resistance, and ice shell resistance (thickness):

$$\theta = \frac{1}{1 + \left(\frac{2\pi k \ell}{\ln D}\right) R} \qquad (1)$$

In Equation (1), the quantity of interest was the overall heat exchanger resistance, R. The thermal conductivity of ice, k, and the length of the heat-intake pipe, ℓ, were known. The pipe-to-air temperature ratio, θ, was calculated from recorded thermocouple data. The ratio of outside-to-inside diameter, D, of the growing ice shell was measured physically at select times during the test period.

Rearranging Equation (1) produces a relationship for R as a function of the temperature and thickness ratios:

$$R = \left(\frac{1-\theta}{\theta}\right) \left(\frac{\ln D}{2\pi k \ell}\right) \qquad (2)$$

The "ice bath" method was a convenient way of determining overall resistance, and it allowed R to be calculated at various levels of heat input (since heat transfer decreases as the insulating ice shell thickens). There was some question as to the relative importance of the liquid-convection contribution to the overall thermal resistance. One would expect the magnitude of the internal resistance to increase with increased ice thickness, since the density differences driving convection decrease. And yet, the total resistance as calculated from Equation (2) at several times during the test maintained a near-constant value. Thus laboratory observations reinforced previous freezing cell experience (8) which had shown internal natural-convection resistance to be secondary to external forced-air resistance[4].

In order to obtain the best assessment of R over time, by minimizing possible inaccuracies in measuring θ and D, Equation (2) was integrated with respect to time over the period of the test. Since near steady-state conditions existed within the ice shell, the transfer of heat into the intake pipe was represented by the following basic heat transfer equation:

(4) In December 1977, three of the fifteen convection cells in the subgrade cooling system were equipped with small submersible-type pumps (that did not interfere with natural convection circulation when turned off) to test the effects of forced circulation around the loop. The long-term result was a less than 1°C lower temperature compared to the unassisted heat exchangers, thus reinforcing the idea that the air-resistance of the cooling heat was the predominant thermal resistance.

$$\dot{Q} = \frac{-2\pi k \ell T_p}{\ln D} = \frac{-2\pi k \ell \theta T_a}{\ln D} . \qquad (3)$$

Eliminating ln D, Equation (2) may be written as;

$$\dot{Q} = \frac{T_a}{R} (\theta - 1). \qquad (4)$$

Integrating Equation (4) with respect to time gives;

$$\int_o^t \dot{Q} d\tau = T_a \int_o^t \frac{\theta - 1}{R} d\tau. \qquad (5)$$

The left-hand integral (LHI) is simply the latent heat of energy of the volume of ice produced during the test period (since heat capacity effects are small and therefore may be neglected):

$$LHI = \frac{\pi d_i^2}{4} (D^2 - 1) \ell \rho L. \qquad (6)$$

Since R was already said to be nearly constant (for the given laboratory conditions), it can be removed from the right-hand integral (RHI):

$$RHI = \frac{T_a}{R} \int_o^t (\theta - 1) d\tau. \qquad (7)$$

The remaining right-hand integral in θ is evaluated graphically after first plotting a curve using recorded thermocouple data. After rearranging Equations (6) and (7), the resulting Equation (8) is used to calculate R, thus:

$$R = \frac{4T_a}{\pi d_i^2 (D^2 - 1) \ell \rho L} \int_o^t (\theta - 1) d\tau. \qquad (8)$$

Final Convection Cell Design

The value of R calculated under test conditions for the parallel arrangement of improved cooling heads was .028°C/watt. However, the final convection-cell cooling head design was somewhat different. Rather than build two types of heat exchanger cooling heads, one pairing two old cooling heads and a second pairing two improved cooling heads, it was decided to mix. Thus the convection cell cooling heads installed at Barrow consisted of one old-type cooling head in parallel with one

new-type cooling head. In addition, a short horizontal section of 10-cm diameter finned pipe was used to feed the parallel arrangement, as is shown in Figure (5).

Figure 5. Cooling head used in subgrade cooling system.

The thermal resistance of the cooling head is sensitive to prevailing wind conditions. Preliminary heat transfer calculations assumed an average wind speed of 20 km/hr for the Barrow area during the winter season. Taking into consideration this average wind speed, and the change in heat transfer area between the convection cell tested in the cold chamber and that actually assembled for the subgrade cooling experiment, a design R value of .012°C/watt was used in predicting expected freezeback characteristics.

The buried heat-intake half-loop consisted of two 5-cm diameter pipes, each 6.4 m in length, connected to one another by a shorter 5-cm diameter pipe, 1.2 m in length. Figure (6) shows in plan the position of the heat-intake pipes beneath the floor of the building.

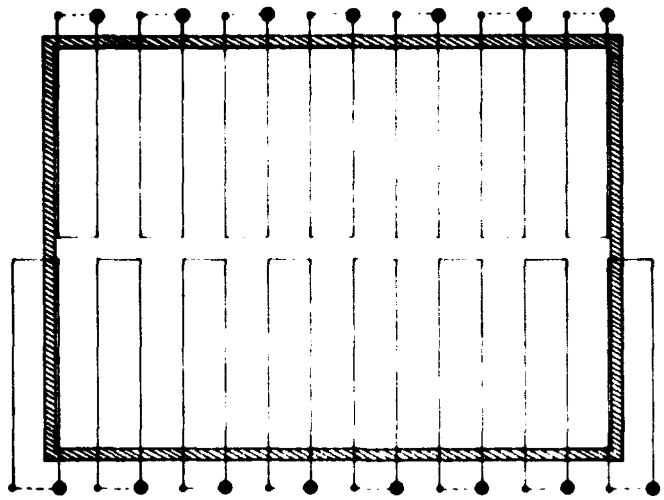

Figure 6. Plan view of heat-intake half-loops beneath the building.

SITE SELECTION AND PREPARATION

Initially, consideration was given to erecting the building on the natural tundra after first placing down a gravel pad of minimum thickness. However, logistically it was more advantageous to make use of an existing gravel pad adjacent to the NARL compound. The results of a subsurface geological investigation performed at that site during April 1976 are documented in Reference (9). In summary, the surficial soils were found to be organic silts with a moisture content averaging 170%, and densities ranging from 960 to 1280 kg/m^3 (60 to 80 lb/ft^3). Massive clear ground ice was found in many locations from a depth of .5 to 1.5 m below the tundra surface, and was presumed to be the result of intersecting ice wedges. This was later confirmed by the excavation of several test pits in the area prior to erecting the building. It was thought that this site, with its high ice content soils, would certainly provide a good "worst case" test of the subgrade cooling concept.

As a first procedure during construction, the building site was cleared of an approximately .6 meter-thick gravel pad to avoid the sloughing of material into trenches cut for the intake-pipe half-loops. The entire cleared site was found to be below the water level of the

surrounding tundra, thus it was necessary to cut a drainage ditch and sump around the perimeter to catch water caused by melting snow, thawing of the active layer and drainage of the stockpiled gravel. Nonetheless, the site became progressively wetter and muddier so that construction at time was very difficult.

The target depth for the heat-intake pipes was about .5 m (1.5 ft) below the tundra surface. The top .3 m of trenched material was found to be thawed, and the frozen soil below this depth was about 50% high ice content organic silt and 50% clear ice (probably the top of ice wedges).(10) The clear ice did not present any clearly defined pattern across the site, thus reinforcing the theory of several generations of intersecting ice wedges. Following the placement of each half-loop, the trenches were backfilled with excavated material and compacted as well as possible. For the most part, the condition of the backfill was saturated or oversaturated soil.

After all 15 intake pipes were installed, the surface of the site was back-bladed by bulldozer to an approximately level grade. Stockpiled gravel was then replaced on the site to an elevation approximately .3 m (1 ft) above the average elevation of the cleared site prior to excavation. The gravel was necessary to elevate the building above summer surface water on the tundra.

BUILDING ERECTION

The nominal 12.5-meter-wide by 16.5-meter-long ATCO "Fold-A-Way" building is a metal structure consisting of 12.5-meter-wide sections, each nearly 3 meters long, which are folded up for compact transport, and erected on site with a light crane. Any number of sections can be joined, and with the addition of endwalls become a completed building.

A crew of six was used to place the panels on a foundation consisting of a wooden groundsill. As each panel was unfolded and placed on the foundation by crane, it was aligned, secured by lag screws to the groundsill, and bolted to the adjoining panel. Figure (7) shows the placement of a typical section. Figure (8) shows the completed structure.

Ground Heating System

The experiment is designed to test the effectiveness of loop-configured convection cells in preserving the frozen condition beneath a heated building constructed at grade. Since the building is not insulated, and need not be heated in its function as equipment storage, it was decided to simulate heat input into the ground by means of thermostatically controlled heat mats buried in the floor. The mats as installed are divided into eight circuits, each on a separate thermostat to better maintain uniform conditions across the floor area. Since areas nearer the wall of the building "see" the cold ground outside, it

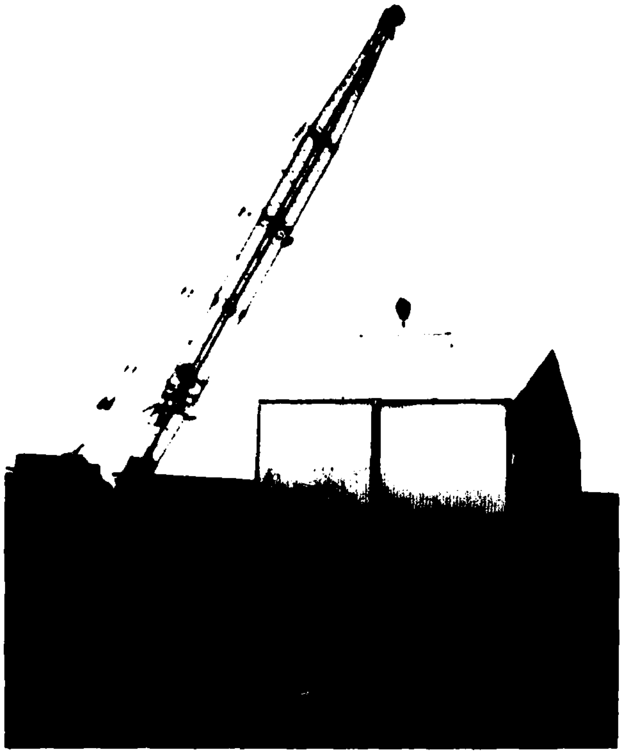

Figure 7. Erection of a typical "fold-a-way" panel.

Figure 8. Completed building with subgrade cooling system.

was thought best to control peripheral mats on the same circuit: thus the network of basically concentric circuits shown in Figure (9) was used.

Remote-sensing thermostats with a small, less than 1°C operating temperature differential, were used to maintain uniform control of the mats. Each thermostat was inserted in electrical conduit and then buried 15 cm (.5 ft) below the heat mats. Two thermostats wired in series were used for each circuit, one set to a slightly higher temperature than the other as backup, should the primary thermostat fail.

In order to minimize the upward loss of heat from the mats to the inside of the cold building, the mats were covered by a 15-cm layer of gravel and a 5-cm thickness of rigid foam insulation. In addition, the foam insulation panels were covered by 30 cm of gravel for protection against vehicle traffic. Pierced steel runway matting (Marston matting) was then placed down as a floor to prevent rutting of the gravel. A cross-section through the subgrade of the building is shown in Figure (10). It should be kept in mind that the building simulates a heated structure on only .3 meters of gravel. Floor grade ("zero" elevation) is therefore defined as the level of the heat mats. The additional gravel and foam insulation above were included only to limit heat loss and provide structural protection.

The convection-cell heat exchanger operates only during those periods when air temperatures around the cooling head are lower than ground temperatures around the heat-intake pipe. The overall yearly effect is a seasonal summer melt beneath the building, followed by a seasonal winter freezeback. Preliminary calculations indicated that it would not be possible to simulate a building with <u>no</u> insulation in the floor. The degree-days of heating throughout the year would be greater than the degree-days of cooling available in the winter. Thus it was deemed necessary to model a floor that was insulated. It was decided to simulate a floor condition whereby maximum summer thaw would not penetrate to the depth of the heat-intake half-loops, and winter freezeback would progress to the elevation of the gravel/tundra interface. The first requirement was thought necessary to guarantee the structural integrity of the heat exchangers.

A trial and error approach was employed, using a heat transfer analysis best described as approximate, since only "average" soil conditions to a shallow depth[5] beneath the building were considered. The following assumptions were made: (1) the convention cells operate during the seven coldest months, and do not operate during the five warmest; (2) the simulated air temperature inside the "heated" building

(5) Since only shallow depths below the floor were considered, the building was treated as an infinite heated plane, and the heat-intake half-loops as an infinite row of cold pipes.

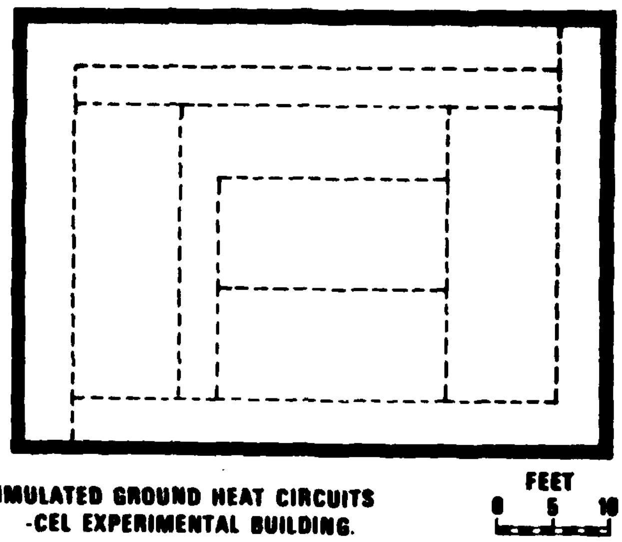

Figure 9. Layout of heat-mat circuits in subgrade.

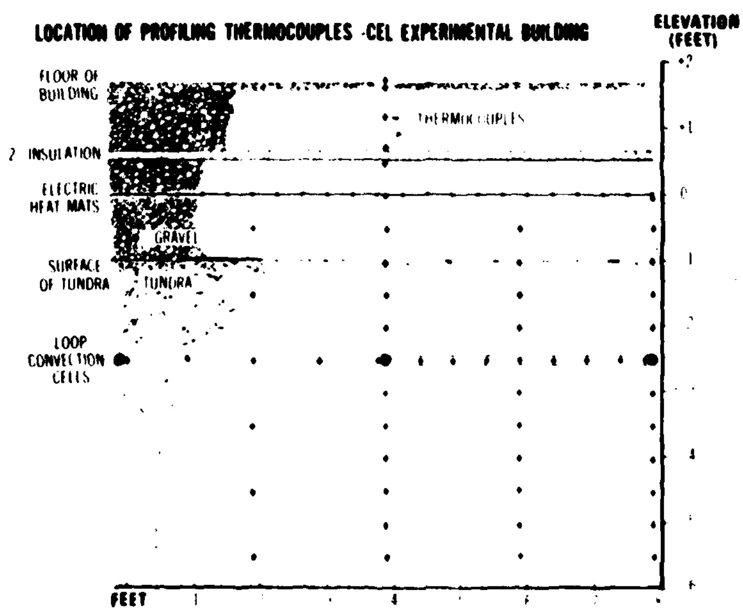

Figure 10. Section through floor and subgrade.

is a constant 20°C; (3) summer thaw begins at the gravel/tundra interface; and (4) the subgrade can be represented by an average soil condition. Regarding this last assumption, results from the previously mentioned geological study, as well as earlier work conducted in the area by Paige (11), were used to generate a "typical" soil condition. It should be kept in mind that a wide variety of soil structures were actually encountered at the site.

The Modified Berggren Equation (12) was used to determine maximum expected penetration of the "active layer" during the five months of convection cell inactivity. Average monthly air temperature data were used to predict freezeback characteristics. Ground temperatures were generated numerically by the super-position of two temperature fields--that due to the presence of the cold intake half-loops (a function of air temperature and thermal resistance) and that due to the long-term presence of the 0°C isotherm. Both temperature fields were referenced to an undisturbed ground temperature of -10°C, which is typical of the Barrow area.

The preliminary trial and error heat transfer analysis indicated that a building placed on a .3-meter-thick gravel pad, and heated to 20°C, would require a floor containing a 5-centimeter thickness of foam-type insulation (thermal conductivity of .03 watt/m-°C). For such a simulation, the summer thaw depth would be approximately .3 meters beneath the gravel/tundra interface. Winter freezeback would send the thaw front back to the gravel/tundra interface. Thus the thermostats were set to model this condition.

Each of the eight electrical circuits was wired to a separate hour-meter so that individual as well as total heat output could be calculated. When energized, the heat-mat network had an energy output density of 172 watt/m^2, with a total energy output of approximately 28,500 watts.

INSTRUMENTATION

In order to monitor changes in convection-cell and subgrade temperature, approximately 150 copper-constantan (type "T") thermocouples are connected to a datalogging thermocouple thermometer. The recording instrument is housed in an insulated, thermostatically controlled box that is heated by four 100-watt light bulbs connected in series-parallel for longer life. Two small fans inside the box insure that air remains at a near uniform temperature. The datalogger is set to record air temperature and convection-cell heat-intake pipe temperatures hourly. In addition, all channels are activated once weekly by personnel from NARL, and the data output is forwarded to CEL for analysis.

Temperature Data

Thermocouples were placed on each of the convection cell heat-intake half-loops, each of the heat-mat circuits and each of the thermostat capillary bulbs. Vertical strings to a depth of 29 feet were placed

in the natural ground away from the building, and at the center of the building. Three additional vertical strings to a depth of 19 feet were positioned at quarter points along both the long and short axes of the building, and under the edge of the building on the short axis. A profiling network of 57 thermocouples at various horizontal and vertical positions was placed around heat-intake pipe "I" and near "G," as is shown in Figure (10). In addition, numerous other miscellaneous thermocouples were installed to monitor such things as air temperature, refrigerant temperature, datalogger "hot box" temperature and thermocouple calibration. Thermocouples placed top and bottom on the 5-cm thick foam insulation were used to estimate the upward loss of heat to the cold building.

Level Data

A permanent benchmark was installed in the frozen ground near the building following recommendations set down by Monograph III-C4 of the United States Army Cold Regions Research and Engineering Laboratory (CRREL). A 6.4-meter length of 5-centimeter diameter pipe (with flange at bottom) was frozen into the ground, the top 3 meters of that pipe being surrounded by a larger, 15-centimeter-diameter pipe casing. The annulus between large and small pipe was filled with a grease to prevent the transmitting of heaving forces to the benchmark.

Foundation level stations were located on the outside edge of the wooden groundsill foundation around the perimeter of the building. In addition, floor level stations were marked at various points on the protective Marston matting inside the building. The location of all level stations is shown in Figure (11). Level readings are taken near the end of each summer and winter season, and whenever CEL personnel are at NARL.

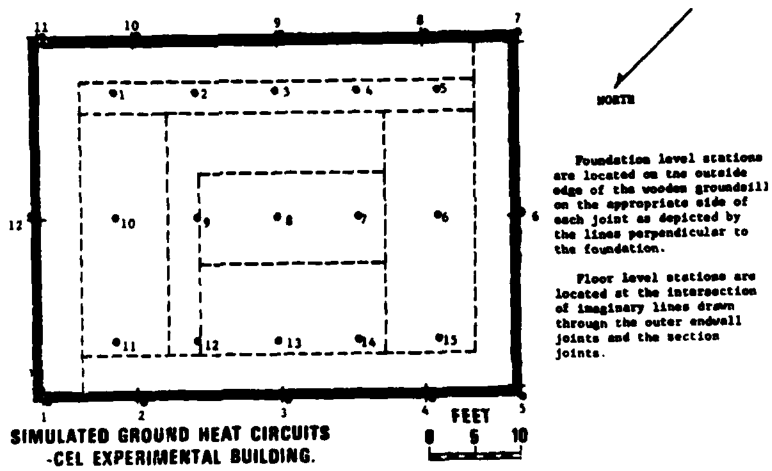

Figure 11. Location of level stations on floor and foundation.

PERFORMANCE

The convection cells were charged in late September 1976 with a liquid refrigerant consisting of 50 percent water and 50 percent ethylene glycol by weight. For the next two months, the heat exchangers were allowed to refreeze the thawed fill material in the trenches around the heat intake pipes. The electric heat mat circuits were energized in early December 1976. The .3-meter-thick gravel pad above the tundra thawed within the first week, and after three weeks the upper 15 cm of tundra was thawed in some places. The thaw front stabilized at a depth of 15 to 30 cm below the tundra surface, depending on the location with respect to the heat-intake pipes.

The static position of the thaw front was at a depth greater than that predicted by analysis. Thermocouple data showed that temperatures recorded at the capillary tubes were too high, thus excessive heat was entering the ground. The 13°C temperature recorded at "floor" grade (zero elevation) more closely modeled a building heated to 20°C with no insulation in the floor, rather than a building with 5 centimeters of foam-type insulation in the floor.

A thermostat similar to those used in the subgrade cooling system was tested in the cold chamber facility at CEL. It was found that calibration was sensitive to the prevailing air temperature (since the thermostats were placed exposed in the cold building). It was therefore necessary to individually reset each of the units at Barrow. This operation was completed in early April 1977, and during the remainder of the month the thaw zone retreated to a position near the desired interface between gravel and tundra.

Figure (12) shows the progression of the thaw front during the 1977 summer season. Maximum summer thaw extended to a depth of about .35 meter below the gravel/tundra interface, as opposed to the .3 meter approximation generated by modified Berggren calculations. Figure (13) shows the freezeback pattern recorded during the 1977/78 winter season. Desired freezeback was nearly complete by the beginning of March.

Temperature Analysis

When a convection cell is tested at constant air temperature and wind speed in 0°C fresh water, the surface temperature of the heat intake pipe decreases slowly as ice thickness increases. For this near steady-state situation, the ratio of intake pipe temperature to air temperature, θ, becomes a good means of establishing and comparing heat exchanger efficiency. Now assume, for purpose of discussion, that the ice shell can be maintained at some static thickness[6]. Then θ will assume a constant steady-state magnitude. If the air temperature is suddenly subjected to a step change (assume there is no growth or decay of ice),

(6) In theory, the ice shell should continue to grow indefinitely in 0°C fresh water, as long as the heat-intake pipe is maintained below the freezing point temperature.

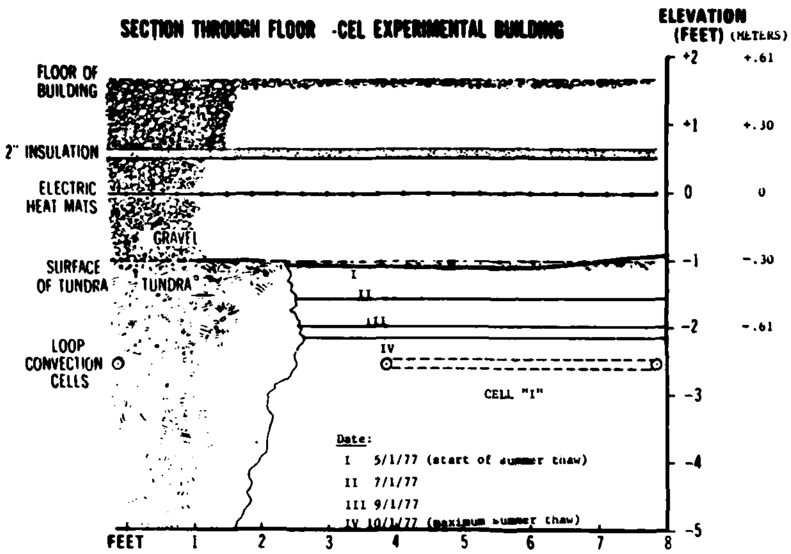

Figure 12. Thaw progression -- 1977 summer.

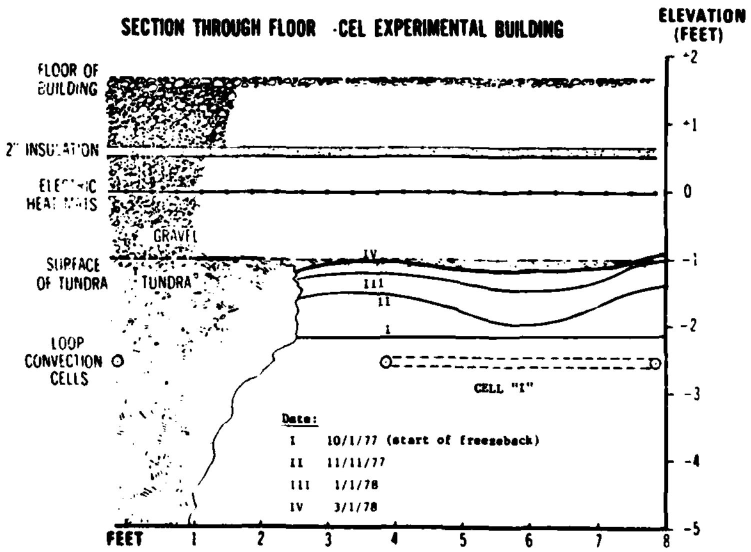

Figure 13. Thaw regression -- 1977/78 winter.

the same value of θ will be established--but only after some period of time. During the interim, heat exchanger temperatures (and thus θ) are influenced by the thermal capacitance of the ice shell and liquid refrigerant. Under these transient conditions, the usefulness of θ as an indication of convection cell performance is greatly diminished.

For example, suppose a convection cell with fixed ice shell thickness is suddenly exposed to a constant warmer air temperature. Depending upon the thermal capacity of the ice mass, temperatures within the heat exchanger will be kept at a lower-than-steady-state level for some time due to the cooling effect of the ice. Thus θ will be "artificially" high. If the cell is exposed to some fluctuating air temperature, it becomes even harder to determine what the steady-state θ should be.

The same problem is true of the subgrade cooling installation where thermal capacity is built into the frozen soil mass. Simply looking at the ratio of pipe-to-air temperature at isolated points in time is largely meaningless because of thermal lag. However, the "true" value of θ can be isolated by measuring both the intake-pipe and air temperatures at regular intervals over a period of time. Figures (14) and (15) show 48-hour recordings of θ, based upon one-hour increments of pipe temperature and air temperature. Wind speed in 3-hour increments is included for reference. It should be noted that the pipe temperature used to generate θ represents the average pipe temperature of the 13 convection cells located totally beneath the building. The two end cells were excluded from the averaging process. Figures (16) and (17) show temperature profiles around the heavily-instrumented convection cell at the start of the December and March test periods, respectively.

The two periods 16-18 December 1977 and 22-24 March 1978 were selected, since they reflect times during which there were no major fluctuations in environmental conditions: wind speed was generally steady, and near the 20 km/hr magnitude assumed as a design average; air temperatures recorded during the preceding days (not shown), were close to those recorded during the respective test periods. In addition, the period in March was thought to be of interest, since it corresponded to the vernal equinox. The nearly equal periods of daylight and darkness at that time produced a more pronounced diurnal temperature fluctuation.

Figure (14) shows how θ appears to climb as air temperature climbs, an effect due to thermal lag. As air temperature increases, the temperature of the liquid refrigerant tries to adjust accordingly, but is delayed by the cool-down effect of the frozen soil mass. Figure (15) presents a similar, but even more pronounced picture. Here the transient θ both rises and falls with the fluctuating air temperature.

It is interesting to note that the transient θ lies consistently between .4 and .6 for both periods. A value of .5 is probably a reasonable approximation for the steady-state idealization. This magnitude

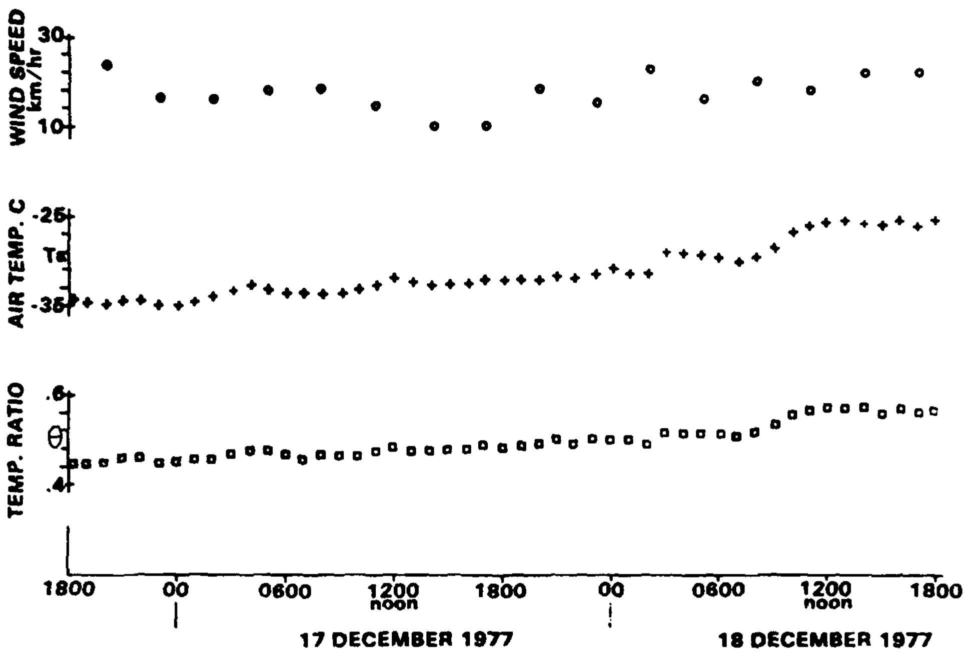

Figure 14. 48-hr convection cell temperature response.

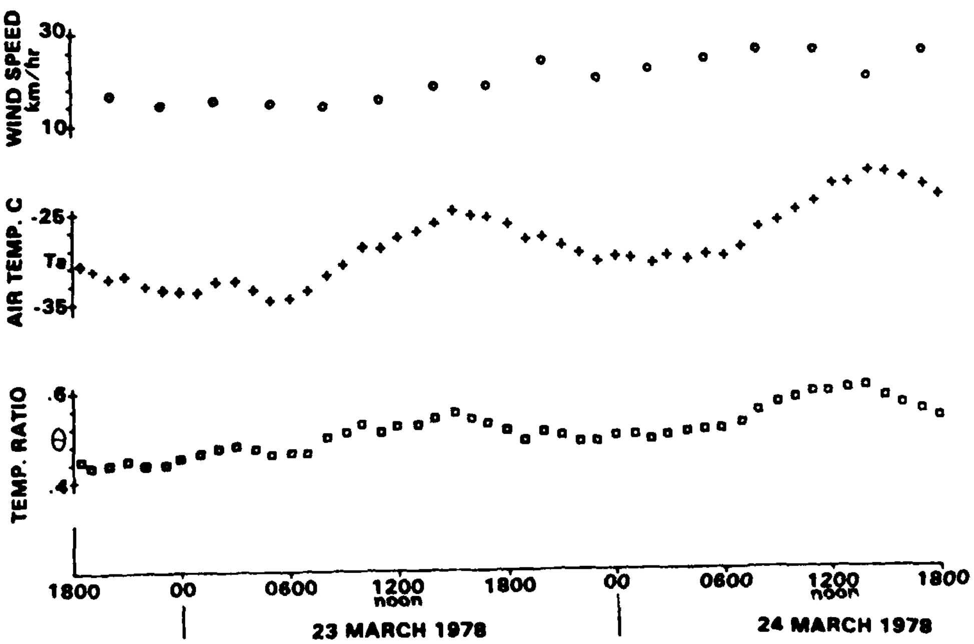

Figure 15. 48-hr convection cell temperature response.

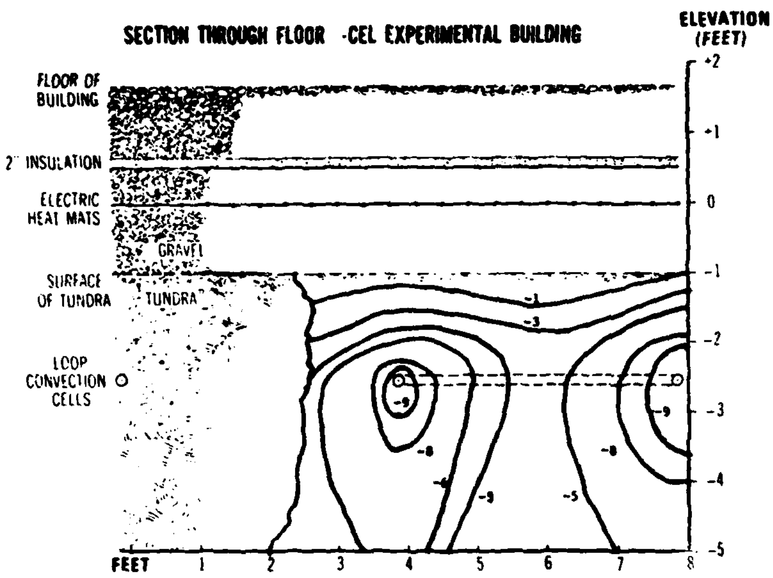

Figure 16. Temperature profile around cell "I" -- 16 December 1977.

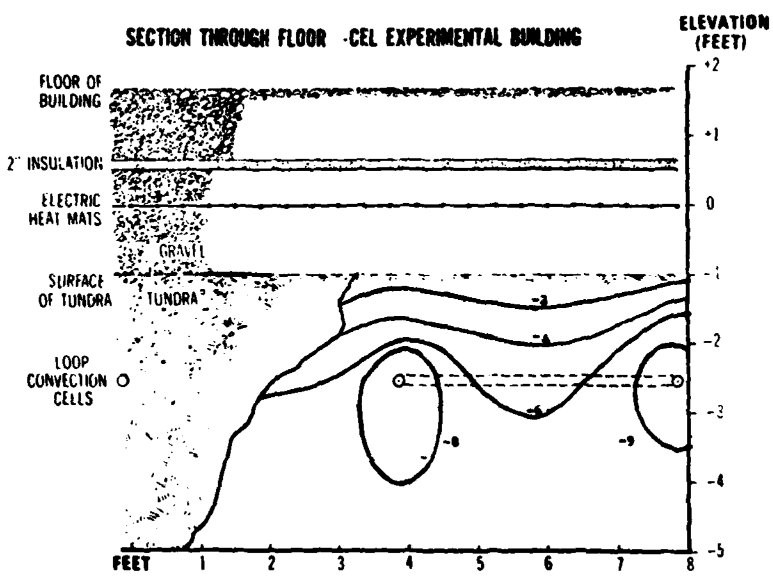

Figure 17. Temperature profile around cell "I" -- 22 March 1978.

is additionally supported by an elementary energy balance involving actual average conditions recorded during the two time periods. To this end, Equation (4) can be used, since it describes the loss of heat from a cooling head (regardless of whether it is in an ice bath or soil). Rearranging Equation (4) to solve for θ gives:

$$\theta = \frac{\dot{Q}R}{T_a} + 1 \; ; \tag{9}$$

where $\dot{Q}$ is the heat lost from one cell. Letting θ represent an average value for the 13 convection cells totally beneath the building means Equation (9) can be rewritten as:

$$\theta = \frac{\dot{Q}_t R}{13\ T_a} + 1 \; ; \tag{10}$$

where $\dot{Q}_t$ is the heat rejected by all 13 cooling heads.

The quantity $\dot{Q}_t$ is the heat actually carried and removed by the subgrade cooling system. It represents the energy produced by the heat mats, less any losses and plus any additions. The major heat loss is upward through the foam insulation, and was calculated to be approximately 15% of the actual heat-mat output. Some change in $\dot{Q}_t$ results from adjustments in ground temperature, but these are small compared to the overall heat transfer. They are ignored in this elementary analysis. During the December period, heat from the electric mats is supplemented by latent heat from the growing freeze front. The latent heat addition is equal to about 5% of the heat-mat output. There were no latent heat contributions during the March period since the freeze front was in a static growth condition.

The combined output of all heat-mat circuits is 28,500 watts. During both two-day periods, the mats were active 50% of the time. This percentage represents a weighted average of all eight circuits. The heat rejected by the cooling heads in December, taking into consideration "on" time, upward losses and latent heat additions, is:

$$\dot{Q}_t = (28{,}500)\ (.5)\ (.85)\ (1.05) = 12{,}700 \text{ watts.}$$

When this quantity is substituted into Equation (10), along with an average air temperature of -30°C and R value of .012°C/watt, the computed value of θ is:

$$\theta = \frac{(12{,}700)\ (.102)}{(13)\ (-30)} + 1 = .61.$$

The heat rejected in March is:

$$\dot{Q}_t = (28,500)\ (.5)\ (.85) = 12,100 \text{ watts.}$$

Evaluating Equation (10) for this heat output and average air temperature of -28°C gives:

$$\theta = \frac{(12,100)\ (.012)}{(13)\ (-28)} + 1 = .60\ .$$

The values obtained by this simple energy balance are consistent and near the range of values obtained by direct graphic representation of temperature data.

Level Data

Table 1 presents level data for points located on the wooden groundsill foundation. Data collected from points located on the floor are similar. As expected, there is limited heaving during the winter freezeback and settling during the summer thaw; however, movement of floor and foundation has not handicapped the utility of the structure.

DISCUSSION

The objective of the experiment at Barrow was to evaluate a subgrade cooling system which would greatly reduce the thickness of gravel required under a heated building. The original intent was to model a structure on permafrost that: (1) was heated to 20°C, (2) contained a floor with a 5 centimeter thickness of foam-type insulation, and (3) was placed on a .3-meter thick gravel pad. This model was never achieved, partially because of the sensitivity of thermostat calibration to air temperature, but mainly because it is just not feasible to model a constant air temperature boundary condition (i.e., 20°C) with the configuration of heat mats used. The capillary bulbs to the thermostats were buried in the gravel pad, halfway between the heat mats and tundra surface. It was necessary to select a single operating temperature at capillary-bulb depth. The thermostats then maintained this temperature by controlling the "on" time of the heat mats. If the air within the building were actually heated, then the temperature at capillary-bulb depth would have to fluctuate up and down as the freeze front periodically retreated and advanced.

As it turned out, the operating temperature selected for the thermostats most accurately modeled the summer season, thus during this time appropriate amounts of heat were released into the ground. As a result, thaw penetration was near the initial design value. As the freeze front advanced upward during the winter season, the deviation away from "model" conditions increased dramatically. During the March 22-24 test period,

Table 1. Foundation elevations referenced to stable benchmark (converted from feet to centimeters).

Level Station	9/19/76	11/21/76	Δ 9/19-11/21	5/14/77	Δ 11/21-5/14	9/5/77	Δ 5/14-9/5	12/9/77	Δ 9/5-12/9	5/19/78	Δ 12/9-5/19
1	-83.2	-81.17	+2.0	-80.53	+0.64	-81.44	-0.91	-78.61	+2.83	-78.30	+0.31
2	-84.4	-82.14	+2.3	-82.36	-0.22	-83.67	-1.31	-81.38	+2.29	-81.44	-0.06
3	-84.4	-82.42	+2.0	-82.14	+0.28	-84.19	-2.05	-82.30	+1.89	-82.14	+0.16
4	-83.5	-81.96	+1.5	-81.05	+0.91	-84.19	-3.14	-83.30	+0.89	-82.97	+0.33
5	-82.9	-81.41	+1.5	-81.41	0.0	-83.39	-1.98	-81.50	+1.89	-81.90	-0.40
6	-86.3	-83.27	+3.0	------	----	-84.67	-----	-81.72	+2.95	-80.83	+0.89
7	-83.5	-81.29	+2.2	-81.01	+0.28	-85.01	-4.0	-82.72	+2.29	-82.66	+0.06
8	-84.1	-81.99	+2.1	-81.29	+0.70	-85.22	-3.93	-83.27	+1.95	-83.51	-0.24
9	-85.3	-83.24	+2.1	-82.97	+0.27	-86.05	-3.08	-84.22	+1.83	-84.43	-0.21
10	-84.1	-81.93	+2.2	-81.66	+0.27	-85.74	-4.08	-81.90	+3.84	-81.75	+0.15
11	-83.8	-80.92	+2.9	-80.68	+0.24	-82.66	-1.98	-79.52	+3.14	-79.40	+0.12
12	-84.1	-81.29	+2.8	-81.41	-0.12	-81.72	-0.31	-81.75	-0.03	-78.06	+3.69

NOTE: Positive Δ indicates heave; negative Δ indicates settlement.

for instance, the average 12,000 watt heat transfer rate was on the order of 5 to 6 times greater than that which would exist for a heated building with insulated floor. It was gratifying to know that the subgrade cooling system could handle this greater heat removal.

Although the heat transfer model (as originally defined) had failed, the experimental subgrade cooling system had been quite successful.

The building at Barrow was located on just enough gravel to elevate it above summer surface water on the tundra. The heat-intake pipes were buried in ice-rich frozen silts and, as a result, there was a small cyclical movement of floor and foundation. Although such performance was not ideal, the settlement which would have occurred, if the massive ice that was present below had thawed, could have been spectacular, to say the least.

Perhaps the next step in foundation construction is to combine the gravel-pad and subgrade-cooling concepts. Phukan, Abbott and Cronin (13) suggest placing self-refrigerated heat exchangers within a gravel pad thick enough to handle summer thaw and winter freezeback. In this way, freeze/thaw action is restricted to a thaw-stable, non-frost susceptible buffer, and seasonal heave and settlement should be held to a minimum.

ACKNOWLEDGMENTS

The author wishes to acknowledge Mr. John E. Cronin, who was in charge of conducting the subsurface geological investigation and erecting the building. His reports have proven vital to the preparation of this paper. The author also wishes to thank Ms Cathleen J. Cavin, NARL technician in charge of weekly data gathering. Her prompt and timely services have been greatly appreciated.

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CRYO-ANCHOR - PROVEN PERFORMANCE FOR ARCTIC FOUNDATION STABILIZATION

By E.C. Cady[1]

ABSTRACT

The heat pipe is a self-contained, passive structure that uses a liquid-vapor fluid cycle to provide high heat transport rates. The McDonnell Douglas Cryo-Anchor heat pipes use anhydrous ammonia as the working fluid and have found wide application in foundation and support stabilization in permafrost regions. The Cryo-Anchor removes heat from the ground during the cold winter months, which results in maintaining (or even increasing) stability of the frozen soil (permafrost) despite the heat addition from structures, pipelines, etc. Maintenance of frozen soil reduces or eliminates problems of pole-jacking, frost heaving, or piling subsidence and settling. Considerable test data have been acquired on Cryo-Anchor installations over the last several years. Data from the Trans-Alaska pipeline are shown and performance of the Cryo-Anchor in preventing utility pole-jacking and preventing building settling are discussed. The superior performance of the passive, rugged, Cryo-Anchor has been demonstrated.

INTRODUCTION

Increasing activity in the northern reaches of the globe, spurred by oil and gas exploitation and the concurrent population increases, has refocused attention on the problems of construction in areas where perennially frozen soil (permafrost) exists. Most construction engineering problems occur in soils that are poorly drained fine-grained sediments, and these problems are associated with freezing and thawing in the permafrost active layer. Seasonal thawing of the active layer acts in two ways to provide differential earth movement resulting in construction problems. First, ice lens formation in the active layer leads to frost heaving and pole-jacking, and second, disturbance of the fragile surface thermal balance leads to progressive melting of the permafrost below the active layer with resultant subsidence. There are several engineering solutions to construction problems caused by these earth movements. First, the problem can be ignored, as is commonly done with inexpensive or temporary structures such as cabins, etc. Second, up or down earth movements can be allowed for by using jacks in foundation design, etc. Third, the frozen soil can be eliminated; this is done in many ways, such as thawing the permafrost, if shallow, or replacing the frost-susceptible soil with gravel or other non-frost-susceptible fill. Finally, the frozen soil can be preserved, as with

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insulation, thermal isolation of heat sources, and active or passive refrigeration. Often, a combination of the above solutions is used.

Passive refrigeration of the soil preserves the permafrost by using the arctic environment. The McDonnell Douglas Cryo-Anchor is a passive refrigeration device which, using no external power, extracts heat from the soil and rejects it to the cold arctic air during the winter. This heat extraction is so efficient that during the summer when the device is inoperative, the reservoir of cold soil built up in the winter will more than compensate for summer warming effects.

The Cryo-Anchor is a low-pressure, liquid-vapor cycle heat-pipe device that uses ammonia as the working fluid. It provides exceptional one-way heat transfer from the soil to the air only. When the air is colder than the soil, the cycle proceeds. When the air becomes warmer than the soil, the cycle is interrupted. The subcooling that occurs during the winter is substantially below the natural levels attained. This compensates for heat transferred into the ground during summer so that a net annual heat removal occurs.

The operation of the Cryo-Anchor is shown schematically in Figure 1. When the air is colder than the ground, the heat from the soil evaporates the ammonia liquid film on the wall. The ammonia vapor rises to the cold radiator section and condenses, rejecting the soil heat to the cold air. The condensed ammonia liquid flows down the wall to the evaporator section, and the cycle continues. A wick, grooves, or special surface preparation is used for efficient distribution of the liquid in the evaporator section. Because the two-phase evaporation/condensation processes inside the Cryo-Anchor are at essentially constant pressure and temperature, the device also provides a uniform temperature along its entire embedded length. This yields a special benefit in mitigating heave forces and preventing pole-jacking, as well as providing uniformly cold temperatures even at the upper portions of a pile in permafrost. It is not a load-bearing device, but its versatility permits it to be used with any type of load-bearing structure. That is, it can be placed inside hollow piles or adjacent to piles to maintain and improve the adfreeze bond, and it can be placed horizontally under a structure on grade or footings to intercept the heat load and maintain the permafrost. It can be installed during construction or added after completion and can (in some cases) be applied to correct a problem in existing structures. It has already been demonstrated in applications to building foundations, pipelines and utility poles. Other applications are limited only by the imagination.

The Cryo-Anchor is completely passive, having no moving parts and requiring no adjustment nor external power to operate. However, it can be used in conjunction with mechanical refrigeration if desired, for example, to achieve a quick freeze-back of a slurried pile. It is factory sealed and tested and requires no field servicing. Leak testing with helium to a maximum allowable rate of 10^{-9} standard cubic centimeters

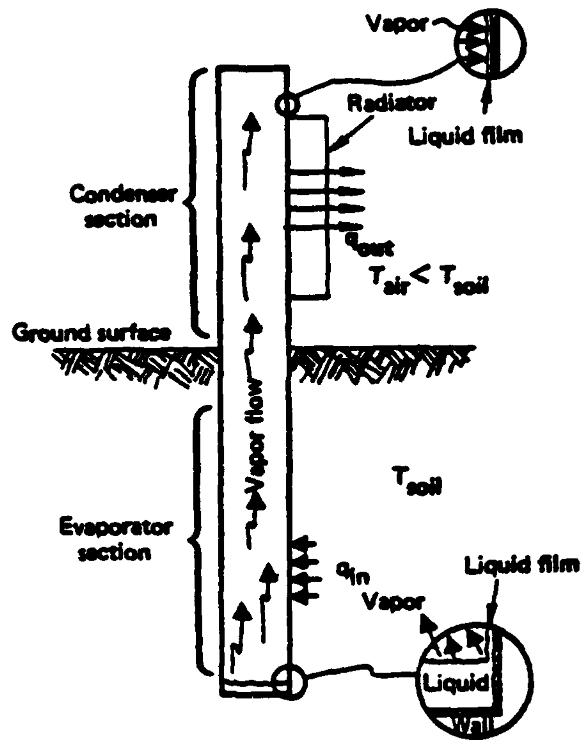

Figure 1. Cryo-Anchor operational schematic.

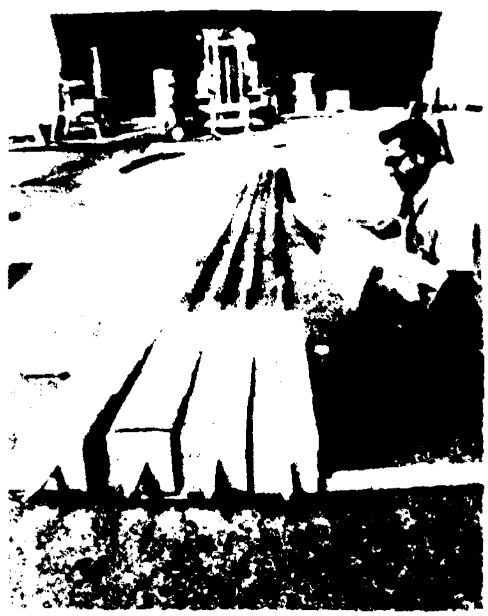

Figure 2. Model 500 Cryo-Anchor configuration.

per second (sccs) ensures that no leakage of consequence occurs over the life of a structure.

CRYO-ANCHOR PERFORMANCE

The Cryo-Anchor uses ammonia as the working fluid because ammonia provides the maximum thermal performance in the temperature range appropriate to the arctic. Table 1 shows the extreme efficiency of thermal transport obtainable with Cryo-Anchors as compared to other mechanisms, including convection and material conduction.

Table 1. Typical effective thermal conductivities of Cryo-Anchor and other transport mechanisms.

	Material or Mechanism					
Effective Conductivity	Cryo-Anchor	Liquid Convection	Copper	Steel	Frozen Soil	Thawed Soil
Btu/hr-ft^2-°F	140,000	2,400	220	25	1.3	0.8

Three basic models of Cryo-Anchor are currently in use: Model 500, Model 100, and Model 800. Model 500, shown in Figure 2, is perhaps the most versatile model. It is made from aluminum tubing with a plate-fin radiator as shown, and can be used for building foundation stabilization in either the vertical or horizontal attitude. The Cryo-Anchors can be placed within pilings, as shown in Figure 3, or adjacent to them. They have been installed horizontally within a slab-on-grade foundation for a school building. Model 500's can be folded for shipment (see Figure 4) which reduces crate size (particularly advantageous for shipment to many remote areas of the arctic). The performance of the Model 500 has been characterized in detail by tests and by computer analysis and is shown in Figure 5. The performance shown is for free convection only (still air) and increases significantly if wind air velocity is available to increase heat rejection from the radiator.

In the application shown in Figure 3, Model 500's were used to stabilize load-bearing piles under existing buildings of the European Space Research Organization (ESRO) Telemetry and Tracking Station located northeast of Fairbanks. In this situation, hollow 4-inch pipe piles buried about 20 feet in the ground were used to support two buildings. For 3 to 4 years after construction and occupancy, no problems were noticed; except that toward the end of this period some differential settling of the building began to occur, and permafrost temperatures next to the monitored piles were very near 32°F. Several alternatives to permafrost recooling were evaluated by field test during the winter of 1971-72. These included passive Cryo-Anchors and active pumped

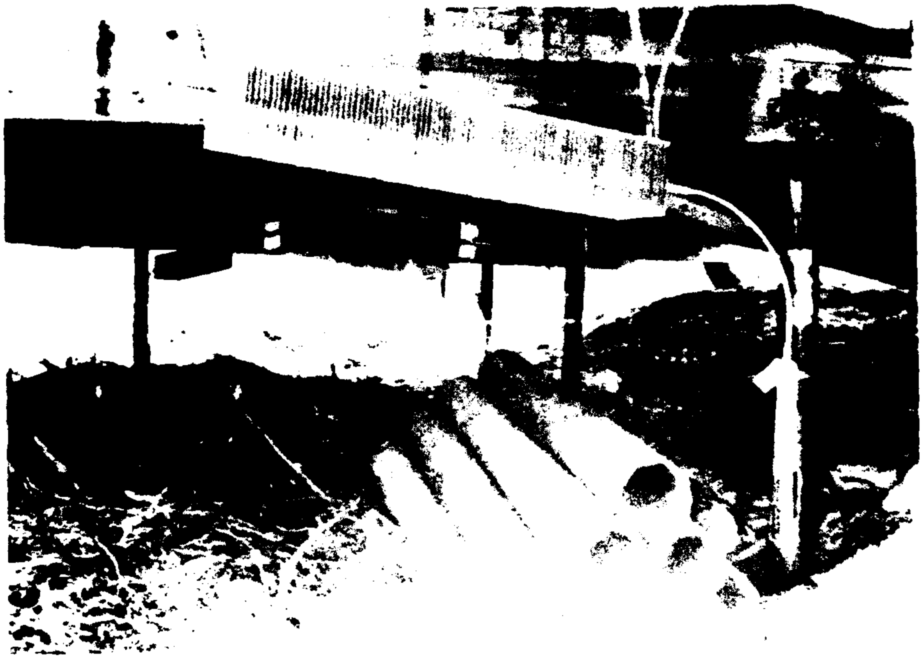

Figure 3. Cryo-Anchor building foundation stabilization.

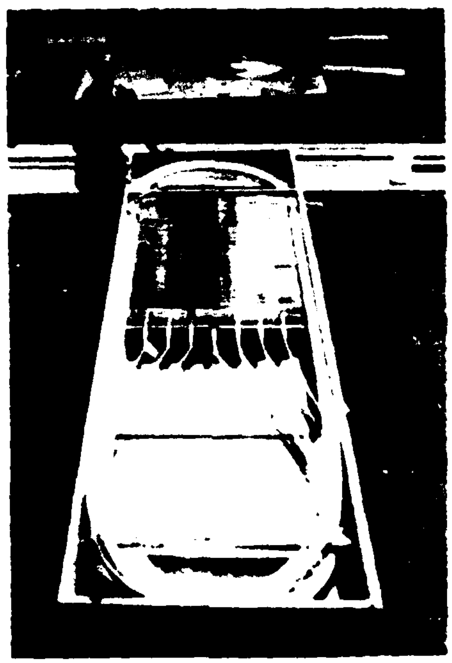

Figure 4. Model 500 Cryo-anchors folded for shipment.

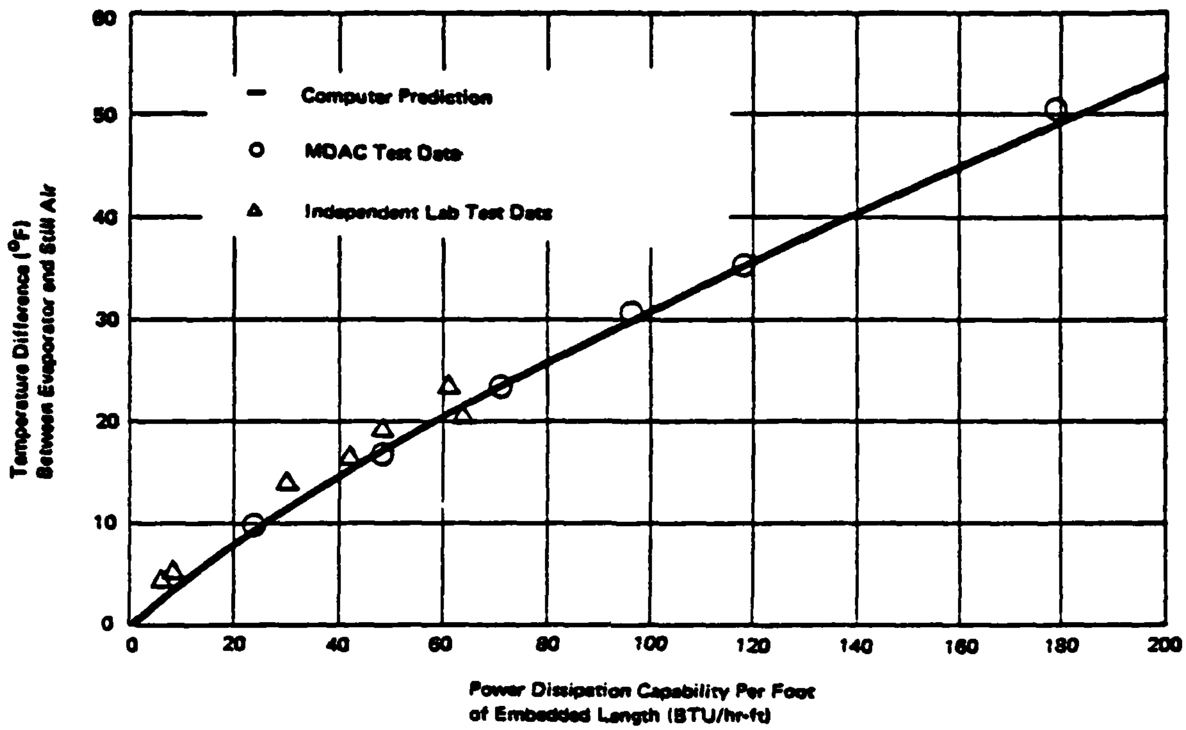

Figure 5. Model 500 Cryo-Anchor performance capability.

liquid loops dumping heat to the winter air, as well as other schemes. From this evaluation, Cryo-Anchors provided the most economical and reliable long-term stabilization of the foundation, and a full-scale installation was completed in September 1972, using a flexible below-ground member inserted into the pipe piles and a horizontal radiator configuration for use under the buildings to accommodate the low head-room available in certain areas.

Figure 6 shows the temperature measured at the bottom of a pile before and after installation of the Cryo-Anchors. The permafrost was warming at about 0.2°F per year, leading to building settlement. After the Cryo-Anchors were installed, the mean and maximum temperatures immediately dropped by over 1.0°F. The building has stabilized and has shown no settlement since 1972.

The Model 100 Cryo-Anchor shown in Figure 7 is a rigid, lightweight aluminum unit that is especially suited for pilings with plenty of

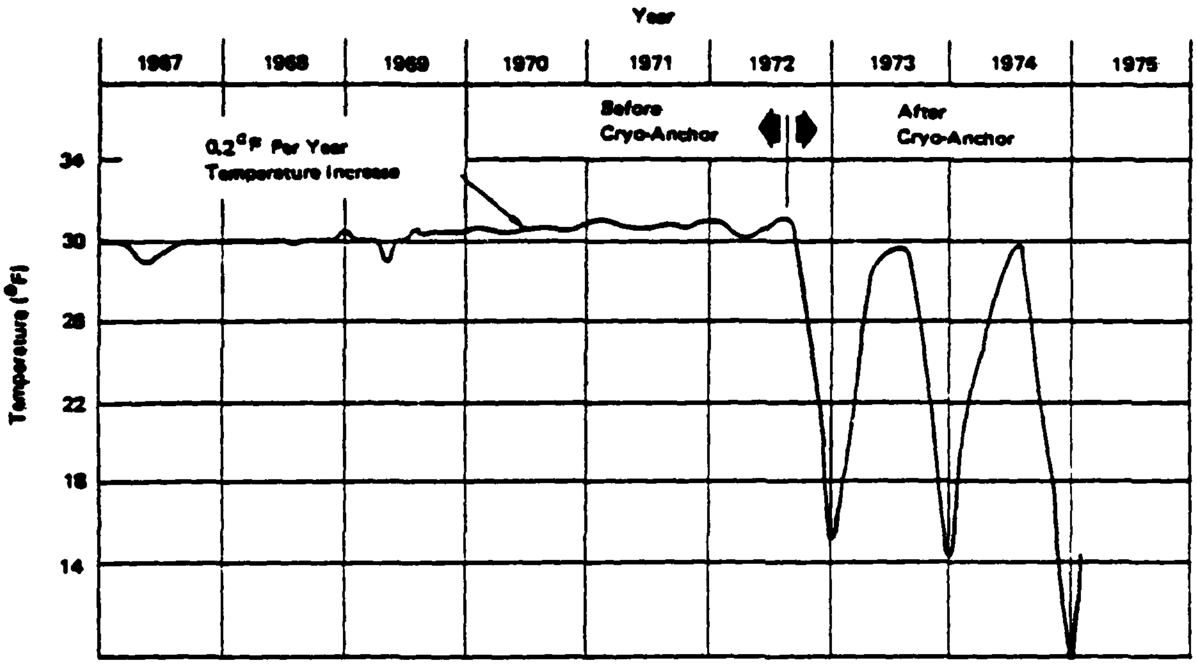

Figure 6. ESRO station Cryo-Anchor performance.

Figure 7. Model 100 Cryo-Anchor for utility pole application.

aboveground clearance, such as telephone or utility poles. The Model 100 is especially effective in preventing pole-jacking of these lightly loaded piles. Figure 8 shows an early test site where four identical test poles were installed initially at the same elevation. Two with Cryo-Anchors have not moved, but the two without protection have moved as seen in the picture. Note the production pole in the background which, although braced, has also jacked.

The pole-jacking mechanism is illustrated in Figure 9. This shows how the adfreeze bond that develops in the freezing soil early in the winter will overcome the pile or pole load, weight, and frictional bond in underlying thawed soil so that the expanding soil will carry the pole upwards. Later, in the thawing season, the same adfreeze bond in the underlying frozen soil will resist the force of the compacting thawing soil. Thus, downward motion of the pole will not occur. Continuation of this cycle, season after season, will raise the pole significantly, and it may actually rise out of the ground.

Figure 8. Pole-jacking prevention using model 100 Cryo-Anchors.

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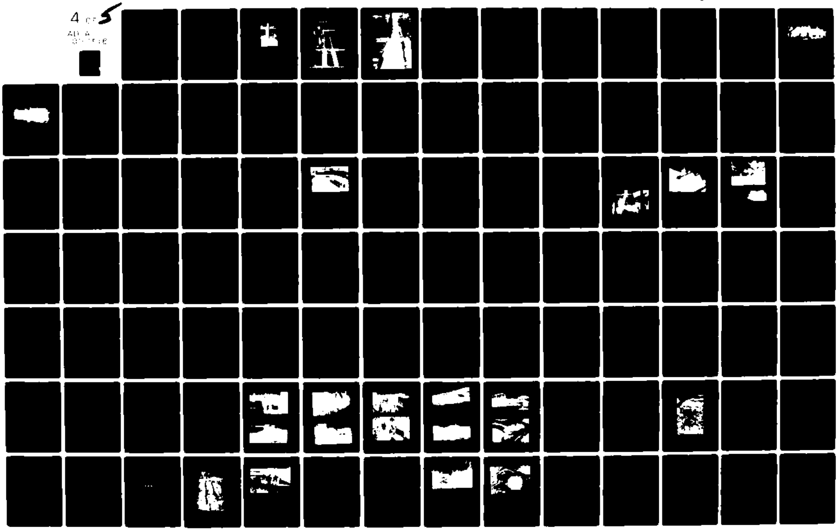

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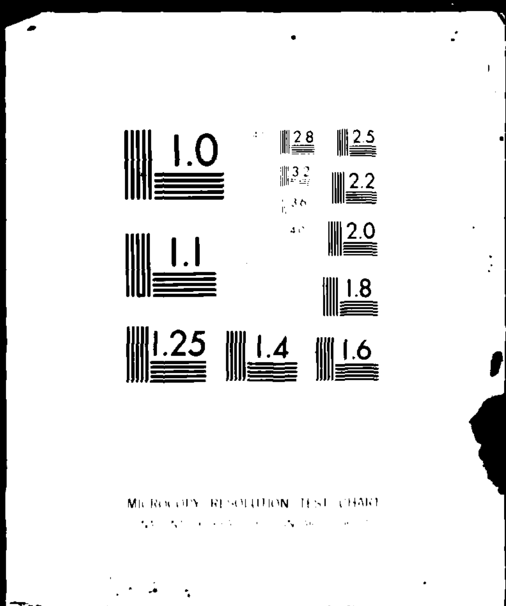


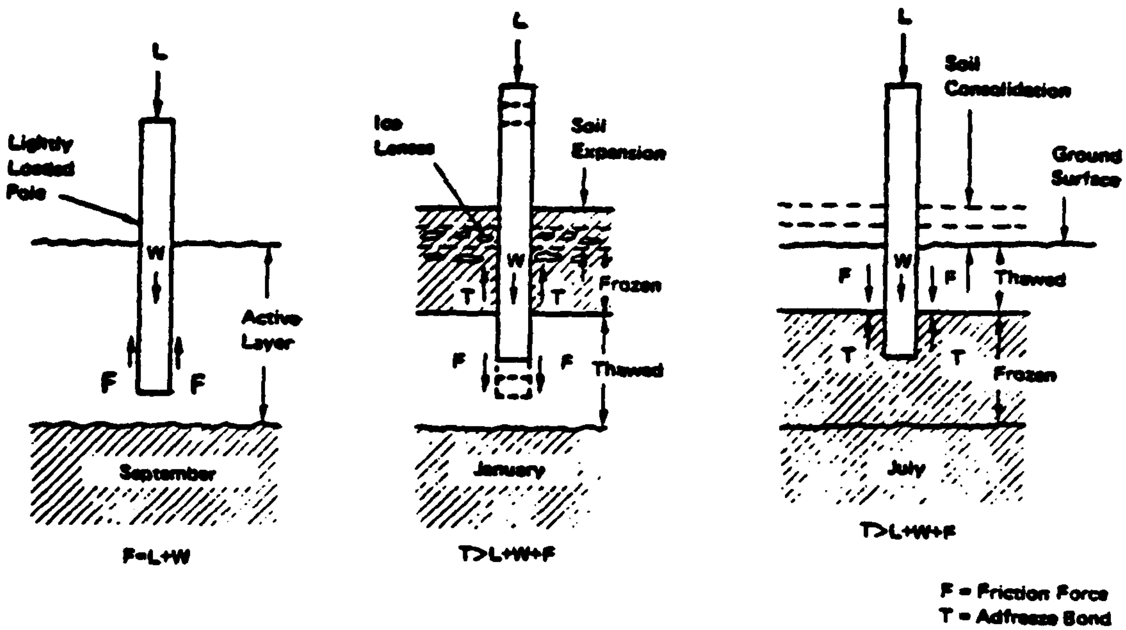


Figure 9. Mechanics of pole-jacking.

Figure 10 illustrates the Cryo-Anchor mechanism for preventing pole-jacking. The figure shows that soil freezing progresses radially outward from the pole at a relatively rapid rate so that ice lenses form in the vertical plane and the soil expands radially, forming a frost bulb. This results from the nearly uniform temperature along the embedded length of the Cryo-Anchor, which provides a nearly uniform force along the pole length. In addition, this rapidly increasing frost bulb provides a large frictional resistance to forces created in the outer regions of the soil. In fact, if there is shallow permafrost, this frost bulb will lock into the permafrost to provide even greater resistance to jacking.

The performance of the Model 100, in still air, is shown in Figure 11, and ignores radiation, which is installation dependent but which could add 40% to the performance shown. A typical installation for a utility pole is shown in Figure 12. Several hundred of these Cryo-Anchors are in use today for utility pole stabilization in Alaska.

The Model 800 Cryo-Anchor, shown in Figure 13, is the unit developed to stabilize the Vertical Support Member (VSM) pilings for the Trans-Alaska Pipeline System (see Figure 14). These Cryo-Anchors are made of carbon steel in lengths to 75 feet, and were made to special

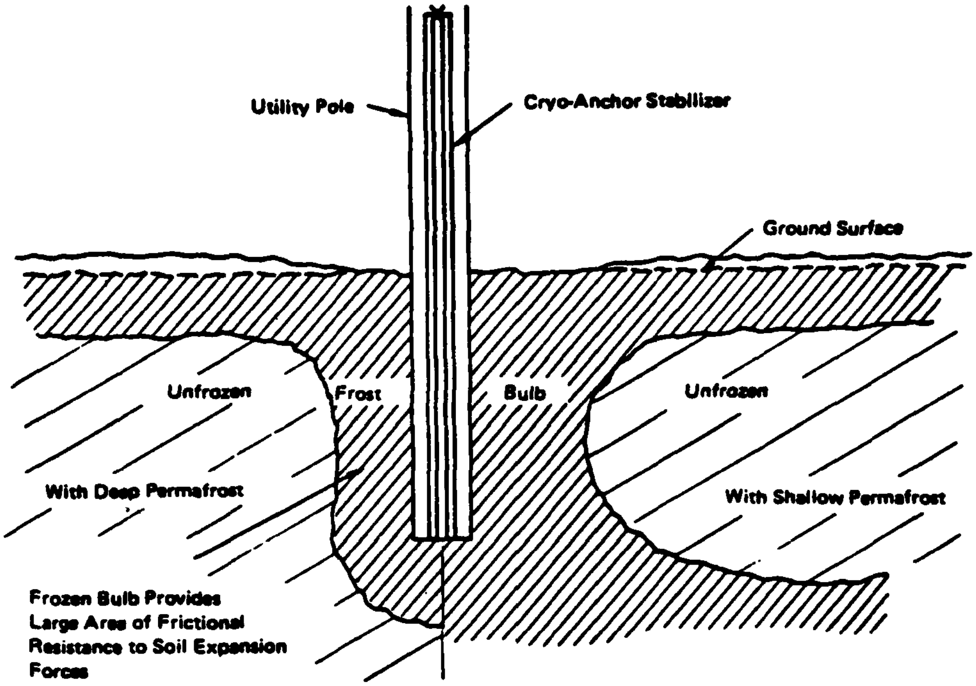


Figure 10. Prevention of pole-jacking through the use of Cryo-Anchors.

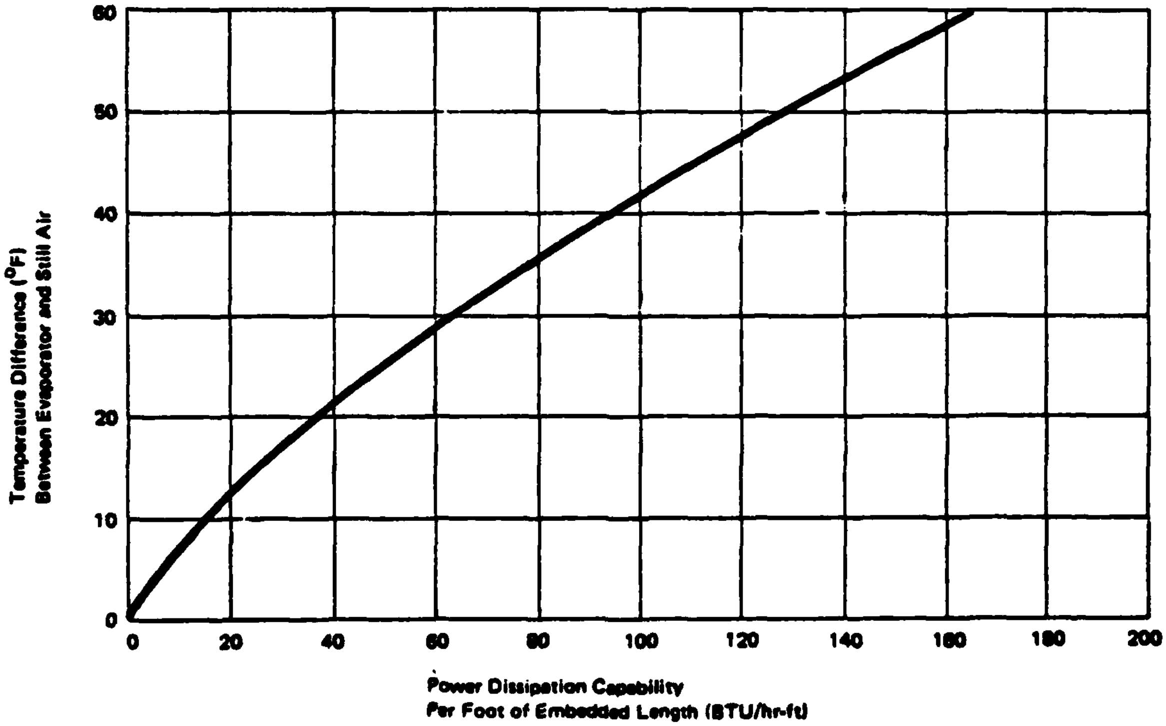


Figure 11. Model 100 Cryo-Anchor performance capability.

Figure 12. Typical model 100 Cryo-Anchor installation for pole stabilization.

requirements for the pipeline. This included a bullet-resistant upper radiator section with 3/4-inch-thick walls, a demonstrated 30-year life, and extreme mass-producibility. The performance of the Model 800 in still air, shown in Figure 15, is for a 34-foot embedded length, a 6-foot radiator, and ignores radiation, which could add over 20% to the performance shown, but which also is installation dependent.

The Model 800's used in the Trans-Alaska Pipeline are incorporated in the VSM's as shown in Figure 16. The thermal performance of the VSM's has been monitored at a field test site near Fairbanks, Alaska, since 1974, as reported in detail in Reference 1. The effectiveness of the Cryo-Anchors in this installation is graphically demonstrated by Figure 17 (from Reference 1) which shows local temperatures at a depth of 20 feet in the slurry around a dummy VSM (containing no Cryo-Anchors) a VSM, and a VSM supporting a valve, both containing Cryo-Anchors. Clearly, there is a significant reduction in the end-of-summer temperatures due to the Cryo-Anchors. This is further illustrated by Table 2, which shows data for 2 years, for several VSM's and for the undisturbed tundra and the gravel pad as well (Reference 1). The Model

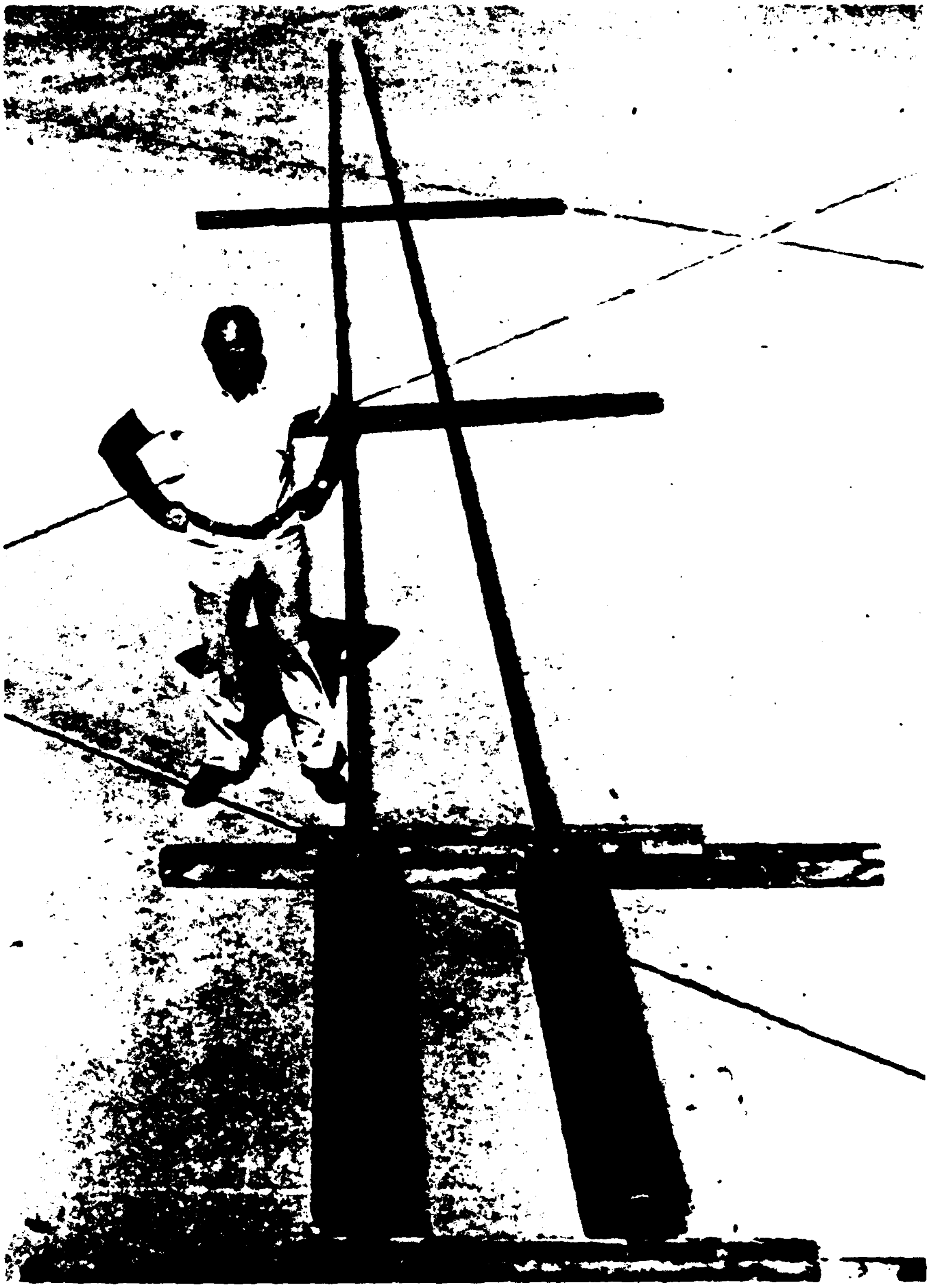


Figure 13. Model 800 Cryo-Anchors for Alyeska Pipeline Service.

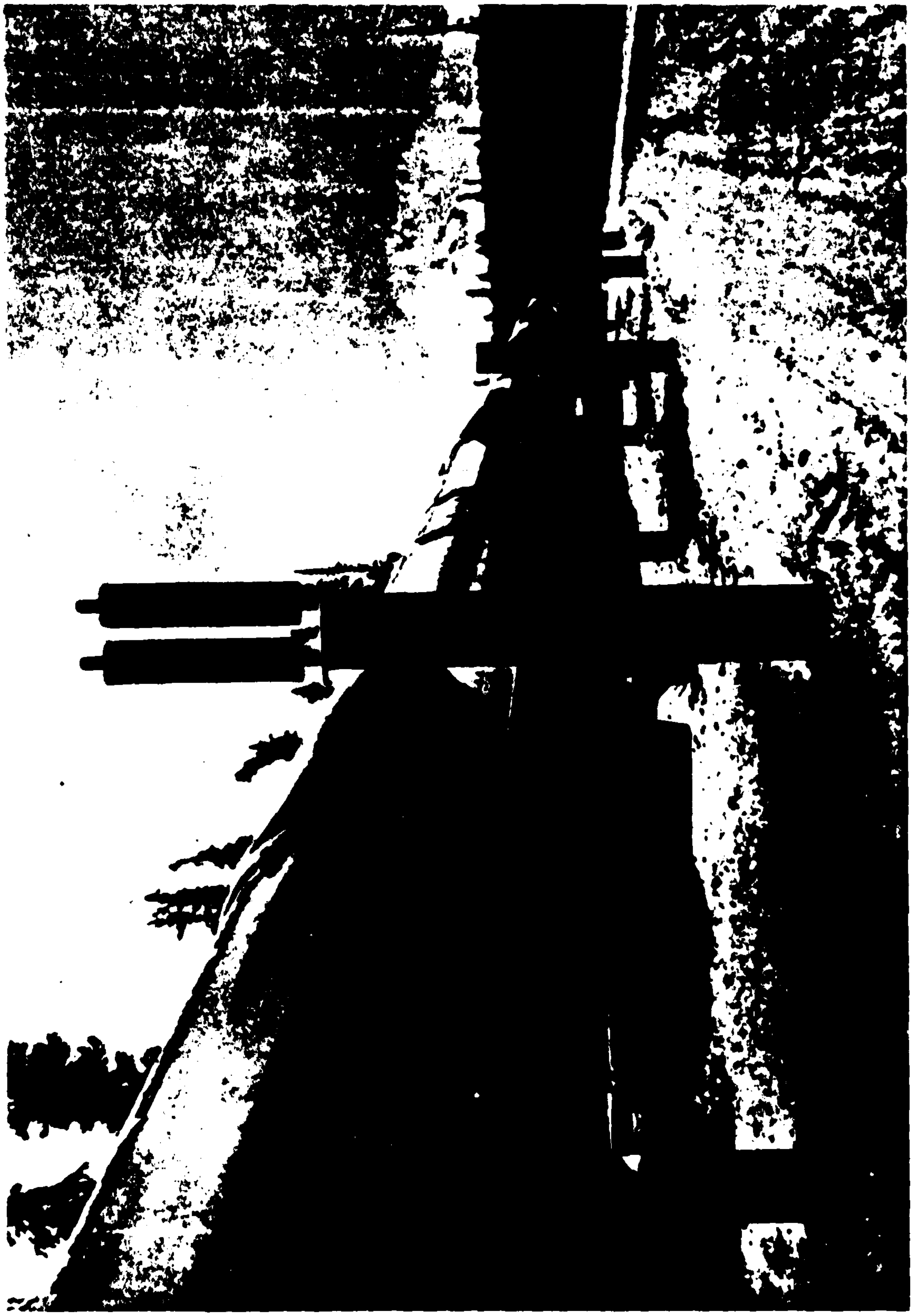


Figure 14. Cryo-Anchor installation on the Trans-Alaska Pipeline.

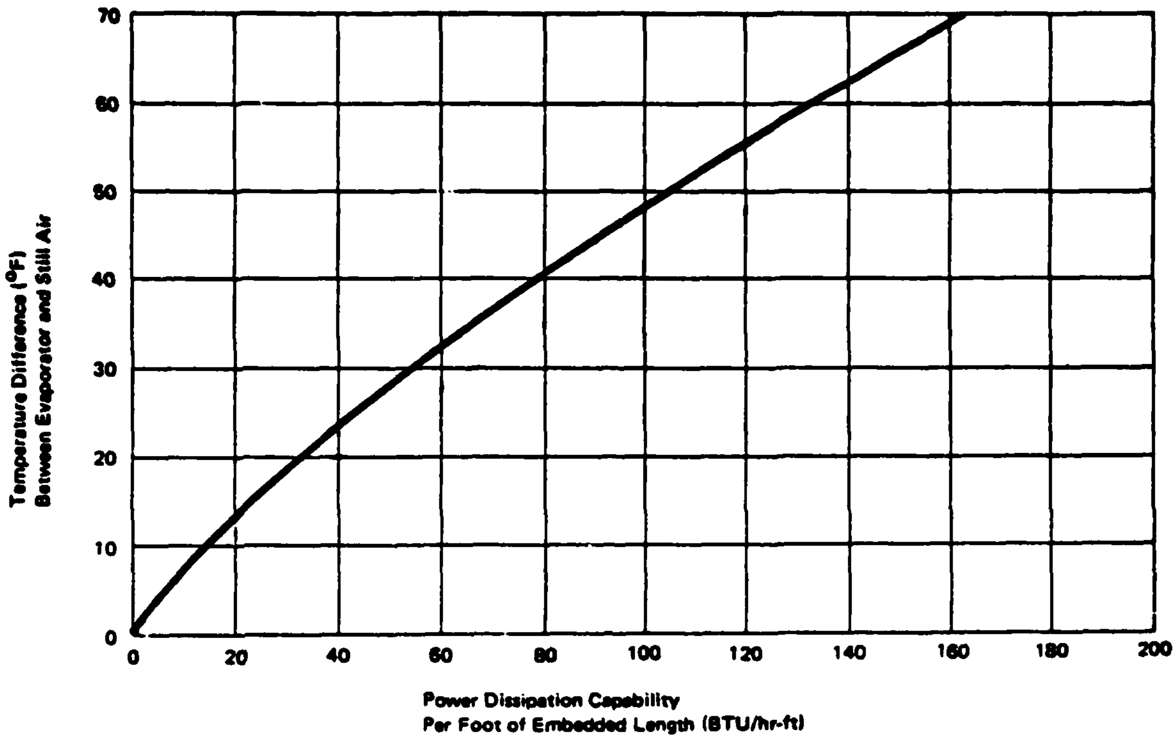


Figure 15. Model 800 Cryo-Anchor performance capability.

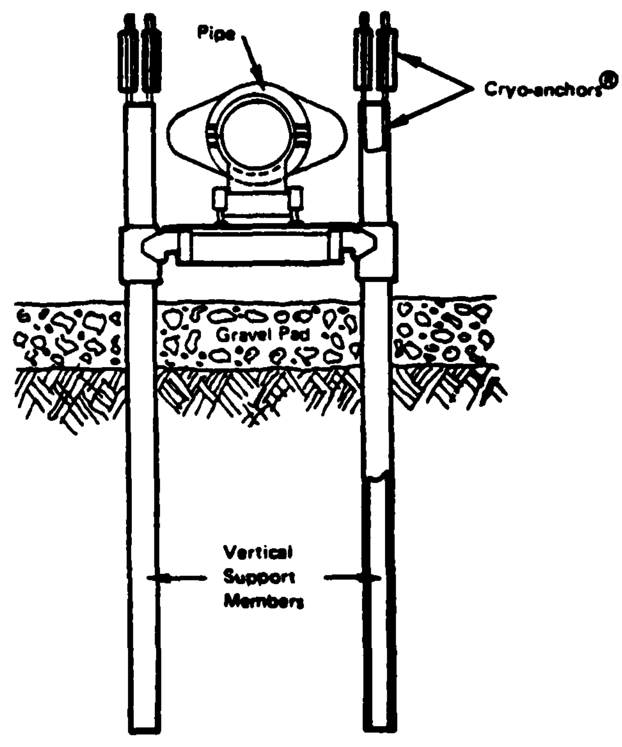
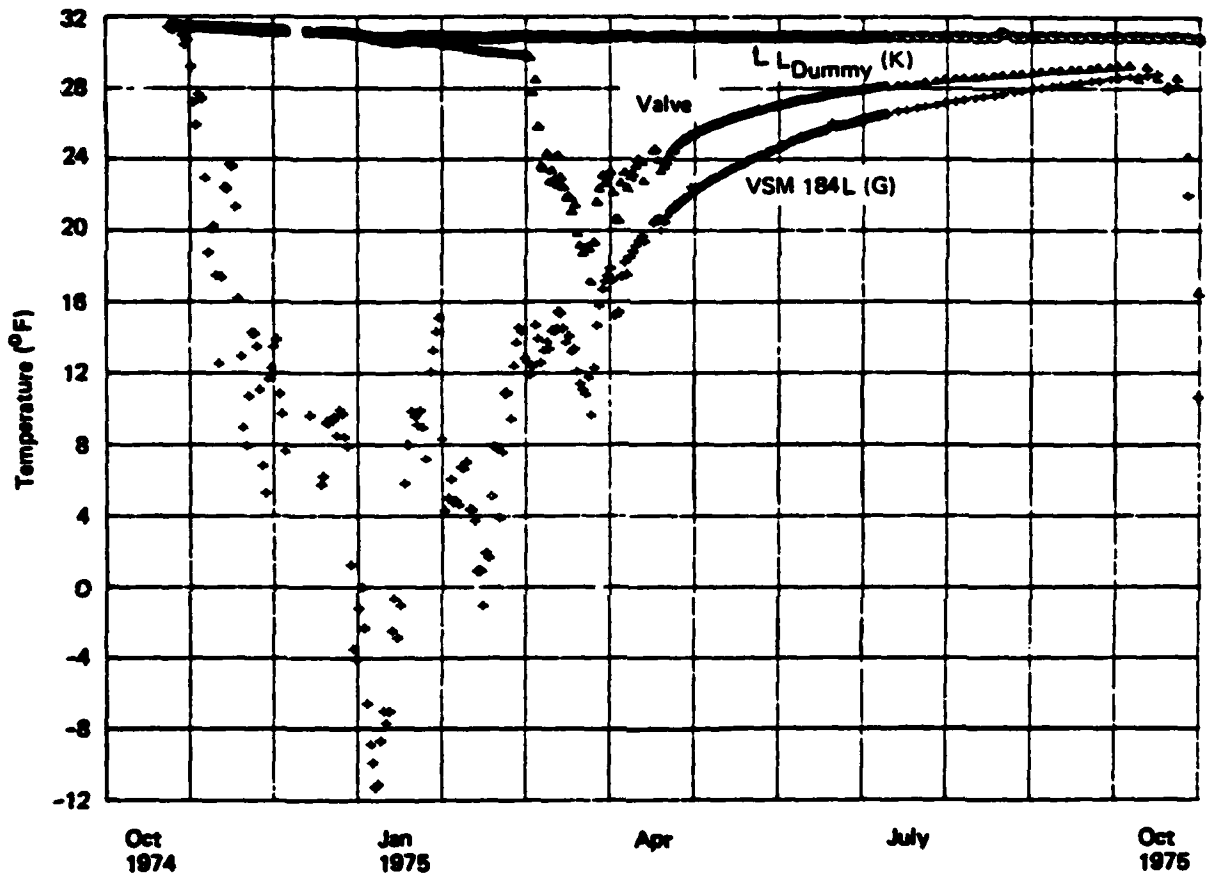


Figure 16. Pipe support bent with thermal vertical support members using Cryo-Anchors.



End-of-Summer Temperatures:
Dummy 31.0°F (No Cryo-Anchors®)
Valve 29.3°F (Cryo-Anchors® installed 3/2/75)
VSM 184L 28.9°F (Cryo-Anchors® installed 10/27/74)

Figure 17. Effect of Cryo-Anchors upon local wall temperatures at the 20-ft depth.

Table 2. End-of-summer temperature summary.

Location	Radius (ft)	Average Temperature**(F) October 27, 1974	October 2, 1975	October 5, 1976
Bent 183***	-5.75	31.17	29.35	28.41
	0.95	31.53*	29.57	28.52
	2.75	31.27*	29.53	28.62
	5.75	31.21	29.60	28.65
Bent 184	-5.75	30.86	28.82	28.44
	0.95	31.10*	28.95	28.46
	2.75	31.10*	28.97	28.53
	5.75	30.83	29.04	28.62
Anchor	0.95	30.99**	29.87	28.95
Valve	0.95	31.51*	29.56	27.58
Dummy VSM	0.95	31.61*	31.01	30.65
	0.95	31.57*	31.02	30.65
Pad	-	31.40	31.14 (10/26/75)	30.92 (10/13/76)
Tundra	-	30.69	30.67 (10/26/75)	30.16 (10/26/76)

*Temperatures probably influenced by VSM installation.
**Strength-weighted average over embedded length below 10-ft depth as measured from local organic or gravel surface.
***One radiator of VSM 183L was insulated during the winter of 1975-76.

800 Cryo-Anchor installations have consistently shown soil subcooling performance in excess of that anticipated from design-analyses, and are completely reliable in this application. Other prospective uses for the Model 800 are for railroad bed stabilization in permafrost and for elimination of frost-heave effects in thawed ground for a buried, chilled-gas pipeline.

CONCLUSIONS

The McDonnell Douglas Cryo-Anchor has proven to be effective for stabilization of structural foundations in arctic regions. They have been in use since 1970, and hundreds are currently in use for a variety of applications, including stabilization of utility poles, building foundation pilings, and even a slab-on-grade foundation. Nearly 140,000 units are in use on the Trans-Alaska Pipeline system, demonstrating superior performance in maintaining the structural integrity of the pipeline supports. Cryo-Anchors prevent permafrost degradation due to heat input from structures, pipelines, or disturbed tundra surface.

They provide increased adfreeze bond for pilings by reducing the frozen soil temperature. Cryo-Anchors are reliable and cost-effective insurance for all kinds of structures against construction problems due to building on permafrost.

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SECTION IV:
CONSIDERATIONS AND TECHNIQUES NECESSARY IN COLD REGION CONCRETE CONSTRUCTION

THERMAL BEHAVIOR IN A CONCRETE STRUCTURE IN INTERIOR ALASKA

By Fred A. Anderson[1]

ABSTRACT

During the summers of 1977 and 1978, the Alaska District, Corps of Engineers, completed construction of a control works structure on the Chena River Project near Fairbanks, Alaska, in the interior or continental climatic zone. The structure contained 17,572 m^3 (23,000 cu/yds) of concrete. Of this amount, 16,041 m^3 (20,983 cu/yds) was placed in the summer of 1977. During the winter of 1977-1978, the internal and external temperatures were measured at regular intervals using embedded and non-embedded resistance thermometers. The temperatures in the mass concrete and exposed structural concrete as monitored by the resistance thermometers were compared to temperatures predicted from a finite element analysis of the structure.

INTRODUCTION

During the summers of 1977 and 1978, the Alaska District, Corps of Engineers, completed construction of the control works structure of the Chena River Project near Fairbanks, Alaska. The control works is a concrete structure located in the Chena River to regulate the flows of the river through an embankment dam which is the major feature of the project.

The control structure consisted of a mass concrete base slab to provide stability (including resistance to sliding and uplift) and a reinforced concrete superstructure which includes control gates, gate operating machinery and fish passageways. The control structure contained 17,572 m^3 (23,000 cu/yds) of concrete. Of this amount, 16,041 m^3 (20,983 cu/yds) was placed in the summer of 1977. Concrete placed in 1977 included all of the base slab and most of the superstructure. The remainder of the concrete was placed during the summer of 1978.

Careful temperature control of all concrete placed was considered essential to reduce cracking in the structure due to temperature differentials in the concrete. Two rates of temperature change were determined to be critical: these were the long term cooling of the interior mass concrete with respect to the more rapid cooling of the exterior mass concrete, and the rapid cooling of the exterior surfaces of structural concrete placed late in the season and then exposed to thermal shock when protective coverings were abruptly removed and the ambient temperatures were very low.

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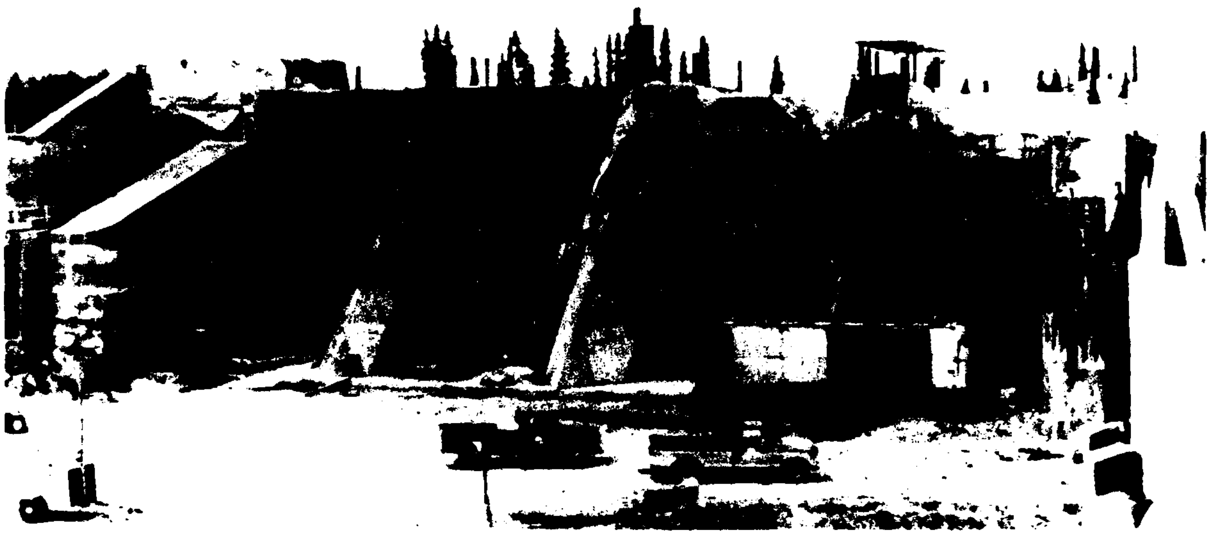

Figure 1. *Outlet works structure as it appeared in September 1977. Plastic enclosure is in place over the superstructure. The Chena River will ultimately flow through the structure from left to right in the photograph.*

The probability that cracking would occur in the absence of effective temperature control was greatly enhanced by the climate in the project area. The project is located approximately 190 km south of the Arctic Circle at 65° North Latitude. The weather in the area is dominated by continental climatic conditions which include great diurnal and annual temperature variations, low precipitation, low cloudiness, and low humidity. The mean annual temperature at the project is approximately -3°C (26°F). During the winter of 1977-1978, ambient air temperatures in the project area reached lows less than -46°C (-50°C). The freezing index in the Fairbanks area is 3060 degree days (°C) (5500 degree days (°F)). Elevation at the top of the structure is 161 meters.

DISCUSSION

Control of thermal cracking in the structure was addressed in the design stages. This work led to specifications requirements which were enforced during construction and then the actual thermal behavior of the concrete was monitored through the winter of 1977-1978 as the

Figure 2. Outlet works structure. The area where the truck is sitting was inundated by rising groundwater shortly after this photograph was made.

heat of hydration was dissipated and the structure reached thermal equilibrium. The final step in the process of predicting and monitoring thermal behavior was a comparison of temperatures measured in the structure to those predicted in the design phase.

During the design phase, the temperature gradients at various locations in the base slab (mass concrete) had been predicted utilizing a finite element analysis (1/, 2/). This method uses a typical section of the base slab which is divided into a grid. The temperature at each intersection of the grid (nodal point) is calculated for set increments in time. This study was conducted to determine the effects of (a) date of concrete placement, (b) amount of insulation (if required), (c) concrete placement schedule, (d) concrete placement temperature and (e) the ambient temperature on the temperature history of the concrete.

Results of the analysis were also used to evaluate the need for embedded pipes in the mass concrete to allow the circulation of cooled or warmed water to allow post-cooling or post-warming of the concrete. Because the cost of installation and maintenance of post-cooling or post-warming systems would have been very high, the intent of the design was to consider those combinations of placing temperature, placing time, and insulation which would eliminate the need for embedded pipes. This was accomplished by adjusting the placement and insulation parameters to maintain a maximum temperature differential within the concrete of 17°C (30°F) occurring over a period of several days. Determination of the maximum allowable long term temperature change was based on an assumed modulus of rupture of 2.4 MPa (350 psi), modulus of elasticity of 2.1×10^4 MPa (3×10^6 psi) and a coefficient of thermal expansion of 10.35×10^6 cm/cm-°C (5.75×10^6 in/in°F). Some creep was also assumed to have occurred. Without an allowance for creep, the maximum temperature differential would be limited to 11°C (20°F). The resulting specification requirements met the above limitations by controlling the maximum temperature of the fresh concrete as it was placed in the forms at 16°C (60°F) and the minimum time between placement of successive lifts at 5 days. These requirements were enforced during construction.

Internal temperatures in the concrete were monitored during the first winter by means of embedded Carlson resistance thermometers. Readings were taken at intervals which increased from daily for the month following placement to monthly at mid-winter, when temperatures stabilized as the heat of hydration was dissipated. Figure 3 shows the locations of the Carlson thermometers.

Placement of the concrete in the base slab was started on 10 May 1977 and was completed on 6 June 1977. Placement of concrete in the superstructure was started on 6 June 1977 and was halted on 6 October 1977. The remainder of the concrete in the superstructure was placed in June, 1978. After 26 October 1977, the base slab was entirely covered with water. The temperature of the water surrounding the base slab was approximately 0.5°C (33°F).

The mass concrete in the base slab contained 3.8 cm (1 1/2 inch) maximum size aggregate. Normally, mass concrete placed by the Corps of Engineers contains larger aggregate sized; however, the aggregates in the Fairbanks area have been emplaced by the Tanana River and very little material is available larger than 3.8 cm (1 1/2 inch). Otherwise, the aggregate is of excellent quality and mostly composed of granite and gneiss particles which are hard, dense and well rounded.

The assumptions used in the finite element program in the design phase were as follows:

Foundation temperature 2.3 m (7.5 ft) deep, constant at 0°C (32°F).

Mean annual temperature - -3°C (-26°F)

Wind - constant at 8.05 km/hr (5 mph)

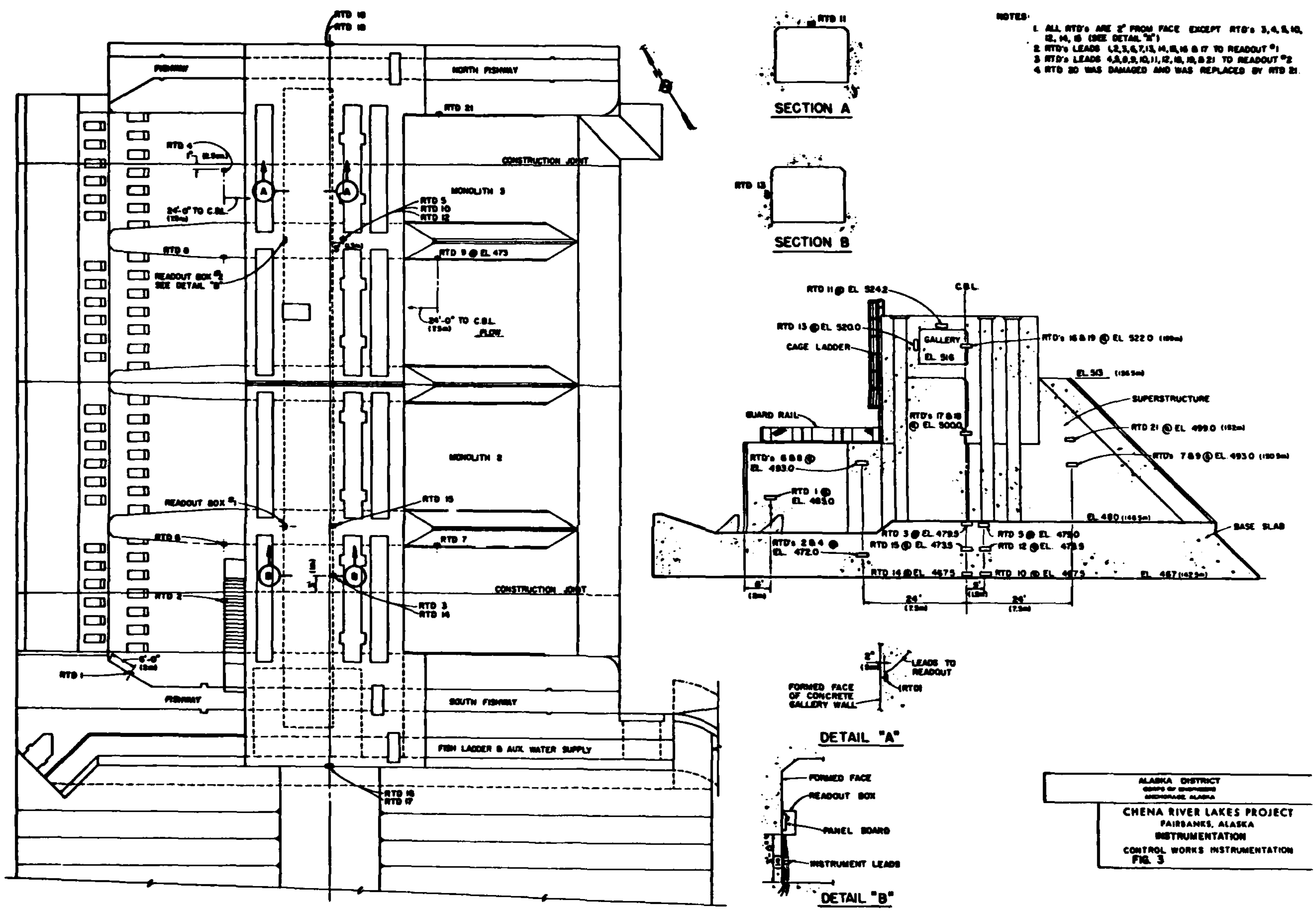

Figure 3. Control works instrumentation.

Concrete placed in two 1.53-m (5-ft) lifts topped with one 0.92-m (3-ft) lift.

Foundation material is gravel

Concrete mix contains 192 kg/m^3 (325 lbs/yd^3) of Type II cement for the first two lifts.

Concrete mix contains 208 kg/m^3 (350 lbs/yd^3) of Type II cement for the third lift in the base slab.

Foundation density - 2211 kg/m^3 (138 pcf)

Concrete density - 2347 kg/m^3 (146.4 pcf)

Foundation conductivity - 3.08 kg cal m/hr-m^2 C° (2.07 Btu/ft-hr-°F).

Concrete conductivity - 2.36 kg cal m/hr-m^2 C° (1.584 Btu/ft-hr-°F).

Foundation specific heat - 1.09 J/g C° (36 Btu/ft^3-°F)

Concrete specific heat - 0.96 J/g C° (0.23 Btu/lb-°F)

Concrete adiabatic heat rise at 28 days - 11.4C° (52.6°F)

A description of the program follows:

PROGRAM DESCRIPTION

Normal high daily mean temperatures occur.
Concrete placement temperature - 16°C (60°F)
Concrete is placed every 5 days
Concrete placement starts 15 May
The foundation is exposed for 30 days prior to placement

The results of the finite element analysis were plotted and are shown in Figure 4. Note that in the design stage, placement of concrete was assumed to begin on 15 May and the base slab was not assumed to be covered with water.

The actual values of the parameters governing thermal behavior varied somewhat from those assumptions which were outlined above. Those that are different follow:

Concrete mix actually contained 184 kg/m^3 (310 lbs/yds^3) of Type II cement for the first two lifts.

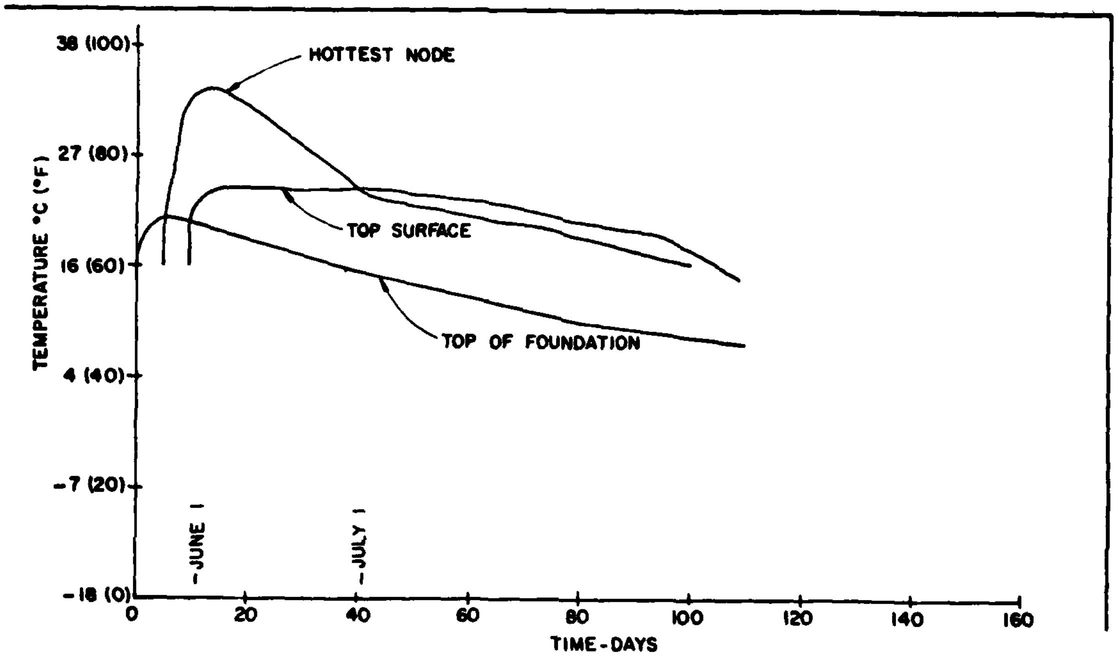

Figure 4. Concrete temperature as predicted by finite element program.

Concrete mix actually contained 262 kg/m^3 (440 lbs/yds^3) of Type II cement for the third lift in the base slab, and all of the superstructure.

Low heat of hydration requirements were specified for the Type II cement and the actual heat of hydration at 7 days ranged from 59 to 68 cal/g. The actual concrete placement temperatures ranged from 10°C (50°F) to 16°C (60°F) with an average of approximately 11°C (52°F).

Concrete placement temperatures were maintained below the specified limit of 16°C (60°F) without utilizing any special techniques for cooling. This was possible because the concrete in the base slab was placed early in the summer before the aggregate stockpiles had warmed up and the mix water from the Chena River was approximately 0.5°C (33°F).

Maximum temperatures at the hottest node as predicted by the finite element program were compared to the maximum temperature at the center

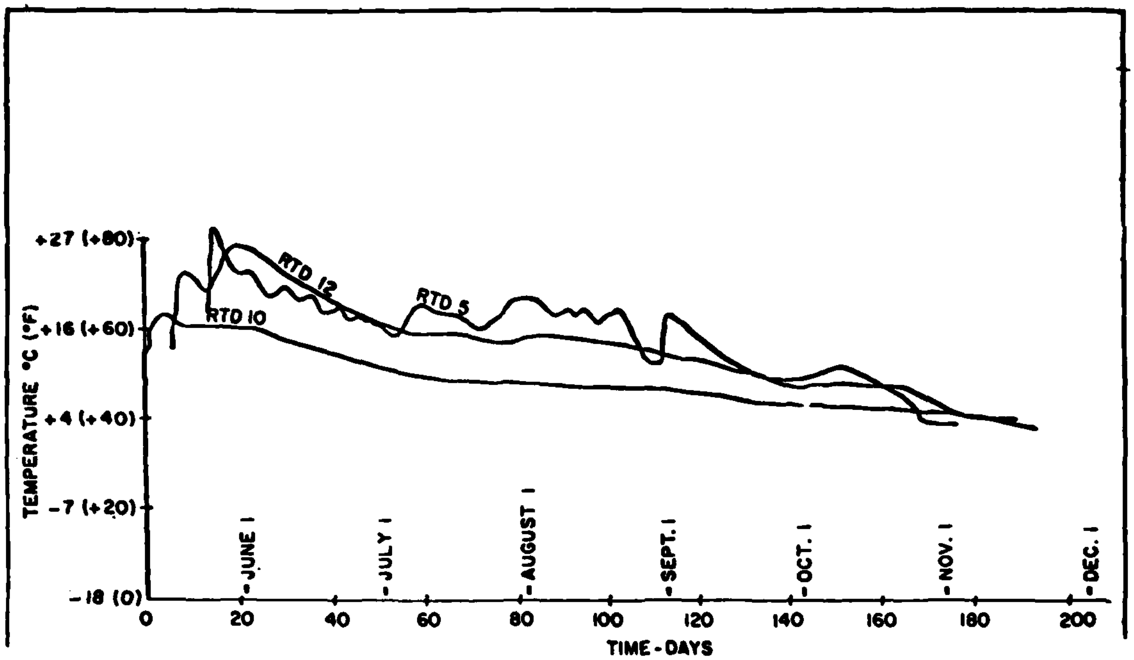

Figure 5. Concrete temperatures as measured by resistance thermometers no.'s 5, 10 and 12.

of the base slab as measured by two embedded resistance thermometers, RTD's 12 and 15.

As a result of this comparison, it was noted that the actual maximum temperatures which occurred were lower than the predicted temperatures by 4°C (7°F) and 7°C (13°F) for each of two locations. The difference between the actual temperatures and the predicted temperatures may be partially explained by the fact that actual mix temperatures averaged 11°C (52°F) while the predicted mix temperatures were assumed to be 16°C (60°F).

From the curves of actual temperatures recorded from the top, center and bottom of the base slab using embedded resistance thermometers, it may be noted that the maximum temperature differential between the assumed hottest node (RTD's 12 and 15) and the coldest point at the

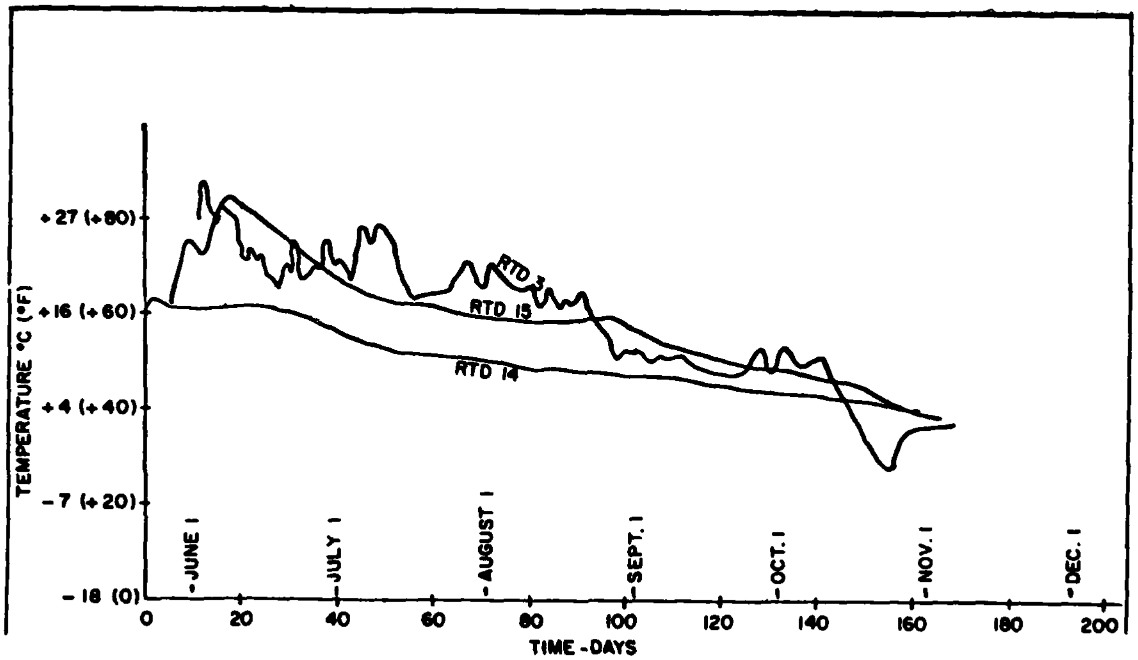

Figure 6. Concrete temperatures as measured by resistance thermometers no.'s 3, 14 and 15.

bottom of the base slab (RTD's 10 and 14) is less than 14°C (25°F) and that the maximum temperature differential between the center node and the top surface is less than 8°C (14°F).

The uppermost thermometers (RTD's 3 and 5) were only 5 cm below the top surface. These thermometers indicated very high temperatures in the first few hours after placement caused by heat of hydration and high ambient temperatures; however, these temperatures dropped rapidly as evening approached and were not considered critical since the temperature changes occurred while the concrete was very fresh and capable of resisting cracking by the relatively high capability for creep exhibited by green concrete.

Because of the Contractor's schedule, the placement of the top lift in the base slab coincided with the time of the highest normal ambient temperatures. As the daily mean ambient temperatures decreased after mid-summer, the temperature curve of the upper surface was roughly

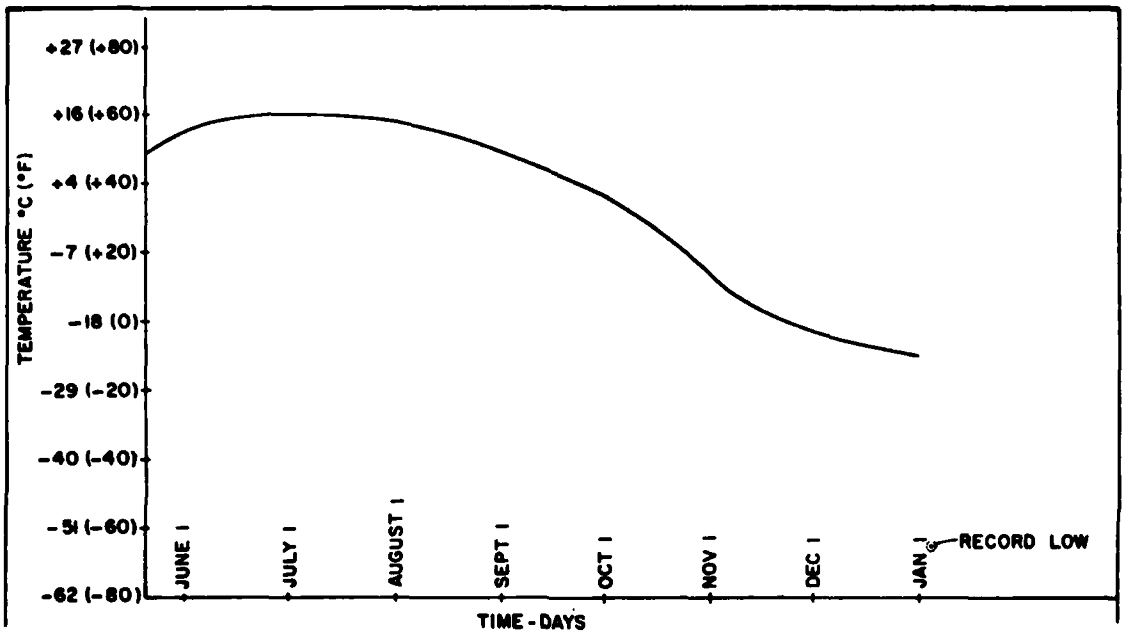

Figure 7. Fairbanks normal mean temperatures.

parallel to the temperatures in the center as the heat of hydration was dissipated, and both were roughly parallel and essentially equal to the plot of average ambient temperatures.

Because temperature differentials were maintained within the predicted levels in the base slab, the temperature control requirements of the specifications were considered to have been adequate. It should be noted that even before the base slab of the outlet works structure was covered with water, the heat of hydration had been dissipated to the extent that the temperature differential between the bottom of the structure and the center was only 2°C.

As below freezing weather began in mid-September, the concern for temperature control was related to the placement of relatively thin sections of reinforced structural concrete in the superstructure. The specifications required this concrete to be maintained at a temperature greater than 0°C (32°F) for a period of 14 days and then insulated

through the winter. Additional research following the award of the contract indicated that the possibility existed for severe cracking of the thin concrete sections, particularly at corners, due to thermal shock at the end of the protection period. An important source of information on this subject was an article by Ram S. Ghosh and J. Neil Mustard 3/ which outlined two major points, the first being that insulation alone would often not prevent freezing in thinner sections, particularly at corners, since the total heat of hydration available was limited by the small amount of concrete enclosed and that protective enclosures or insulation should not be rapidly removed in very cold weather since cracking would then be induced by the contraction of the rapidly cooling outer surfaces.

An important consideration in designing the thermal protection for the superstructure as compared to the base slab was the absence of protection by the water which covered the base slab. All of the base slab is below river level and ground water levels in the area are closely related to the Chena River which will ultimately be diverted to flow through the outlet works structure.

As a result of these considerations and discussions with the Contractor, it was decided to increase the temperature requirements to insure adequate strength development. Protection was provided by a heated plastic enclosure. The temperature within the enclosure was maintained at 13°C (55°F) by ducts which were fed from an oil fired heater located in a trailer van adjacent to the structure. Warmed areas were monitored so that no concrete was exposed to temperatures higher than 16°C (60°F) to minimize the possibility of excessive drying shrinkage or carbonation shrinkage.

Selection of the curing temperature within the protective enclosure was based on work by Klieger 4/ which indicated that a curing temperature of 13°C (55°F) was optimum for strength gain in concrete mixed with Type II cement.

During the protection period, the concrete was moist cured. Following the 14-day period of protection, the Contractor was required to reduce the temperature within the enclosure at a rate such that the temperature differential between the surface of the concrete and the surrounding air did not exceed 14°C (25°F). The limitation on the temperature differential was determined by Ghosh and Mustard based on the thickness and shape of the concrete structure.

As an added precaution, insulation was maintained over two of the more massive pier sections after the protection period was completed.

SUMMARY AND CONCLUSIONS

In northern areas having a continental climate, the wide range of temperatures and the short transitions between summer and fall

require careful control of the thermal behavior of mass concrete and adequate protection of structural concrete if cracking is to be controlled.

High labor and materials costs in these areas force consideration of the control of the placing temperatures, time of placement, volume of placement and insulation in lieu of the post cooling and post warming techniques commonly utilized in warmer climates.

In the Chena Lakes Project Control Works, use of a finite element approach allowed the rational formulation of placement schedules and maximum placement temperatures for the mass concrete which prevented unacceptable temperature differentials. Enclosure and extended protection of structural concrete has proven helpful in preventing cracking, but must be accomplished without creating rapid temperature changes which can induce thermal shock.

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REGULATED-SET CONCRETE FOR COLD WEATHER CONSTRUCTION

By Francis H. Sayles[1] and Billy J. Houston[2]

INTRODUCTION

In general, portland cement concrete placed and cured in a cold environment requires special protection until it develops sufficient strength to resist damage from freezing. Common methods for protecting newly placed concrete against freezing include: preheating the forms before placement, erecting and heating temporary enclosures for placement and curing, heating the aggregate and mixing water, insulating the forms to use the heat of hydration, heating the freshly placed concrete, or specific combinations of these techniques. Each of these methods requires the expenditure of more time and energy than would be required for concrete placed in above-freezing temperatures.

To reduce this increase in energy and time and to extend the construction season for unprotected concrete work, special cements and cement additives have been developed. These concrete materials include: high-early-strength cement, rapid hardening alumina cement, accelerating admixtures, calcium chloride and other additives. However, although these materials greatly reduce the protection time required for concrete, they do not eliminate it.

A cement that gains full strength shortly after placement at below freezing temperatures would be ideal for cold weather construction because it would completely eliminate the need for protection. But, since such an ideal cement that is economically feasible for large concrete structures is not yet available, the next best alternative for cold weather construction is a cement that gains strength rapidly and produces enough heat through chemical reaction to protect the concrete from freezing during the setting period. One cement that has the potential for fulfilling these requirements is regulated-set cement (reg-set).

Regulated-set cements produced in the United States under patents issued to the Portland Cement Association (PCA) can contain from 1 to 30 percent by weight of Calcium Haloaluminate, having a chemical formula $11\,CaO \cdot 7Al_2 \cdot CaX_2$ in which X is a halogen. Reg-set cement contains less dicalcium silicate than ordinary portland cement and the tricalcium aluminate in ordinary cement is replaced by calcium fluoroaluminate. The fluoroaluminate hydration contributes considerable strength to a concrete immediately after it sets and at above freezing temperatures it requires a retarder to control the setting time. Citric acid and calcium sulfate hemihydrate are used as retarders; however, citric acid is used most commonly.

Hoff et al. (1975) reported the typical chemical analysis for the USA produced reg-set and the Japanese equivalent, "Jet-Set," cements to be:

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(2) Research Materials Engineer, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Constituents	Japanese cement[1] A	Japanese cement[1] B	USA cement A	USA cement B
SiO_2	13.7	13.9	17.1	15.0
Al_2O_3	10.8	11.0	10.4	10.2
Fe_2O_3	1.7	1.7	0.9	2.1
CaO	58.6	58.6	60.9	58.1
MgO	0.7	0.8	1.0	1.9
SO_3	11.0	11.1	6.0	6.2
Ignition loss	1.0	0.2	2.7	4.2
Total	97.5	97.3	99.0	97.7
Insoluble residue	NG[3]	NG	0.3	1.11
Na_2O	0.6	0.6	0.64	0.44
K_2O	0.4	0.4	0.16	0.98
Total alkalies as Na_2O	0.86	0.86	0.80	1.08
F	0.9	1.0	NG	NG

(2) After Osborn and Smith (1974),
(3) NG = Not given,
(A and B are cements from different manufacturers.)

As previously mentioned, one possibility for using concrete in below freezing temperatures is to use a cement that not only hardens quickly, but also liberates sufficient heat soon enough to prevent the concrete from freezing (until it has gained sufficient strength to resist damage from such freezing). The listing below gives some average data for the heats of hydration reported by Lea (1971), and others, for different types of cements.

Cement Type	Heat of hydration at 21°C (cal/g), Age, days: 1	3	7	28	90	365
Normal (Type I)	-	61	80	96	104	109
Rapid-hardening (Type III)	-	75	92	101	107	113
Low heat (Type IV)	-	41	50	66	75	81
ASTM (Type II)	-	47	61	80	88	95
High alumina[4]	85	86	87	--	--	110
Regulated set[5]	78	109	115	118	--	--

(4) After Robson (1962)

(5) U.S. Army Engineer Waterways Experiment Station data.

This table shows that regulated-set cement liberates more heat during the first day than the rapid hardening cement does over a period of three days and that during the first seven days it supplies more heat to the concrete than any of the other cements do for an entire year. The combination of an early liberation of heat and early gain in strength makes it feasible to use regulated-set cement at below freezing temperatures.

Regulated-set cement concrete has been used at above-freezing temperatures for some time [(Hoff (1975), Collum et al. (1978)]; however, it has only recently been tested in the laboratory for possible placing and curing at below freezing temperatures [Hoff et al. (1975)].

Also reported by Houston and Hoff (in press) is the testing of regulated-set concrete for two experimental slabs cured out-of-doors in ambient temperatures below -6.7°C, without providing any protection against freezing.

The purposes of this paper are to summarize the results of the previous laboratory and field testing of regulated-set cement for use in cold weather construction and to present a comparison of the construction procedure and the performance of a regulated-set concrete slab with that of a similar size slab constructed of type II concrete in below freezing temperatures.

The laboratory testing program and the results of the field tests will be reviewed first. The construction and performance comparison of a floor slab for a warehouse will then be described.

LABORATORY TESTING OF MORTAR AND CONCRETE

A. Mortar Testing

The initial laboratory testing was performed on mortar mixtures to evaluate various parameters. Both type III (high early strength) and regulated-set cement were used in the mortar tests. Tests on neat cement showed that the regulated-set cement had a setting time of 10 minutes at 23°C while the type III cement set in 3-3.5 hours in an ambient air temperature of 23°C. Other parameters evaluated in the laboratory were varying water cement ratios, admixtures, the sequence of adding materials to the mixer, temperature of the materials, mixing temperatures and curing temperatures, and lengths of curing.

The results of this testing as reported by Houston and Hoff (1975) are:

1. Water-cement ratio - as the water-cement ratios increased the compressive strength decreased. This is to be expected in normal mortars but is perhaps more pronounced when the mortars are exposed to freezing temperatures during curing, as there is more water available for freezing to disrupt the mortar structure.

2. Type of cement - regulated-set cement gained more strength when exposed to -9.5°C during curing than type III high-early-strength portland cement. This is due principally to the rapid set and strength development of the reg-set cement plus the extra heat liberated which sustains the strength development longer.

3. Admixture;

a. Time of set retarder - citric acid appeared to be more effective in retarding the setting time of reg-set cement than plastiment, (water reducing retarder)

b. Air-entraining admixture - this admixture appears to be of some benefit in producing strength gain in reg-set cement mortars cured at -9.5°C. This result is to be expected as air entrainment is known to reduce freezing damage in concrete

c. Water-reducing admixture - this agent appears to be slightly more effective than air-entraining admixtures in producing a gain in strength because it reduces the water requirement which benefits strength development and reduces the amount of water available for freezing in the concrete.

4. Sequence of adding materials to mixer - regulated-set cement should be added in the mixer as the last ingredient, to allow a little more working time before stiffening of the cement begins.

5. Mixing and curing temperatures - lower temperatures produced lower compressive strengths. If mixing water, or cement, or sand were at elevated temperatures before mixing, hydration would begin sooner and sustain itself longer by internal heat production with resulting higher strengths in ambient temperatures of -9.5°C.

B. *Concrete Testing*

Using the information obtained from the testing of regulated-set cement mortar, a typical concrete mix was selected for evaluation. The results of the laboratory investigation of this mixture have been reported by Hoff et al. (1975) and only a summary of the results will be given here. The concrete mixture used contained regulated-set cement, 19 mm maximum size limestone coarse aggregate, and limestone sand. Saturated-surface-dry batch weights were as follows:

Material	Saturated-surface-dry batch weghts, kg
Cement	69.2
Fine aggregate	175.1
Coarse aggregate	256.6
Water	36.7

The mixture contained a measured cement factor of 297 kg/m^3, a water-cement ratio of 0.53, entrained air content of 8.5 percent, and a slump of 63.5 mm. All materials and molds were stored at 1.7°C until immediately before to mixing. Mixing and molding were done at 23 $\pm$ 1°C. The cement was introduced into the mixer as the last step, and the concrete was mixed for 1-1/2 minutes.

This concrete was tested to determine how its strength is affected by delaying its exposure to a -9.5°C temperature after casting, and the effect of specimen thickness on heat and strength development. A summary of these tests indicated the following:

1. The effect of holding the regulated-set concrete at a temperature of 21°C for different periods of time before the concrete was exposed to -9.5°C temperature is shown in Figure 1. An examination of the curves in this figure reveals the following:

Curve	Temp (°C)	Time
A	21±1.5	–
B	−9.5	24 hrs
C	−9.5	3 hrs
D	−9.5	1 hr
E	−9.5	immediately

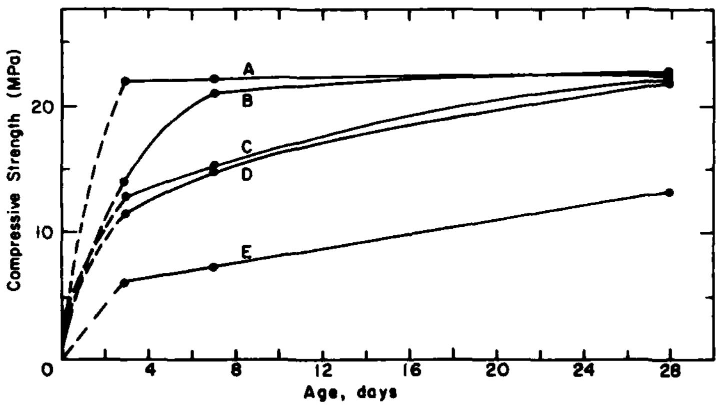

Figure 1. Compressive strength vs. concrete age for concrete specimens exposed to -9.5°C temperature after curing at 21°C for different periods of time [Houston and Hoff (1975)].

a. Concrete specimens mixed at temperatures from 1.7 to 4.4°C and placed in a temperature of -9.5°C immediately after casting had 3-day strengths of slightly less than 6.8 MPa and continued to gain strength while exposed to -9.5°C temperatures until they reached a 28-day strength of about 60% of that for the control specimens which were cured at 21°C - (Compare curve E with A for the control specimens, Fig. 1),

b. The longer the mixture remained above freezing before exposure to the -9.5°C temperature, the greater the subsequent 28-day strength was - (Compare curves B - E),

c. Specimens exposed to a temperature of 21°C until they were 24 hours old and then exposed to -9.5°C exhibited almost as much strength after seven days as did companion specimens exposed at 21°C the full time - (Compare curves A and B),

d. Specimens cured at -9.5°C temperature with one hour or more delay before freezing, gained strength more slowly than control specimens, but at the 28-day age they were practically as strong as the specimens exposed at room temperature the full time - (Compare curves B, C and D with A).

2. In determination of the effect of temperature increase in the concrete caused by hydration of the cement and the transfer of heat to the subgrade, four test slabs were cast. The slabs were 0.51 by 0.51 m square, one was 7.5 cm thick, two were 15.2 cm thick, and one was 30.5 cm thick. One of the 15.2 cm thick slabs was used as a control and was cured at 21°C. Ten centimeters of air dry sand were compacted in the bottom of each mold to represent the subgrade except for the control slab which had no sand base. The molds and sand were brought to -9.5°C before concrete was placed in the midpoint of each concrete slab and 2.5 cm deep in the sand base beneath the midpoint of each slab.

The control slab was placed in an air temperature of 21 ± 2°C with 90 to 100 percent relative humidity, and all other slabs were placed in -9.5°C temperatures immediately after casting. After the temperature measurements were completed and the slabs had cured for 7 days, they were sawed into 15.2-cm cubes, except for the 7.5-cm-thick slab which was sawed into 7.5-cm cubes. These cubes were tested for compressive strength at 8 and 28 days.

The curves in Figures 2 - 4 show that the heat of hydration raised the temperature at the midpoints of the slabs. Temperatures of the slabs with thicknesses of 7.5, 15.2 and 30.5 cm were raised to 8.1°, 14.4°, and 20.5°C, respectively, in 1 to 2 hours while exposed to an ambient air temperature of -9.5°C. After this rapid initial rise, the temperatures were then gradually dropped. This indicated that enough internal heat was generated to sustain the hydration. Curves for the control slab, which was cured in a humid room at 21°C, show the temperature rose to a maximum of 26.4°C in 36 hours and reached the ambient temperature in approximately 72 hours. The midpoint of the other slabs reached ambient air temperature at different times depending upon their thicknesses. The thinnest one (7.6 cm thick) reached ambient air temperature in about 40 hours and the thickest one (30.5 cm) in about 85 hours.

The temperature curves for sand at 2.54 cm below the bases of the slabs rose to a maximum of 14°C under the 30.5 cm thick slab in 5 hours, 11°C under the 15.2 cm thick slab in 2.3 hours, and 6.4°C under the

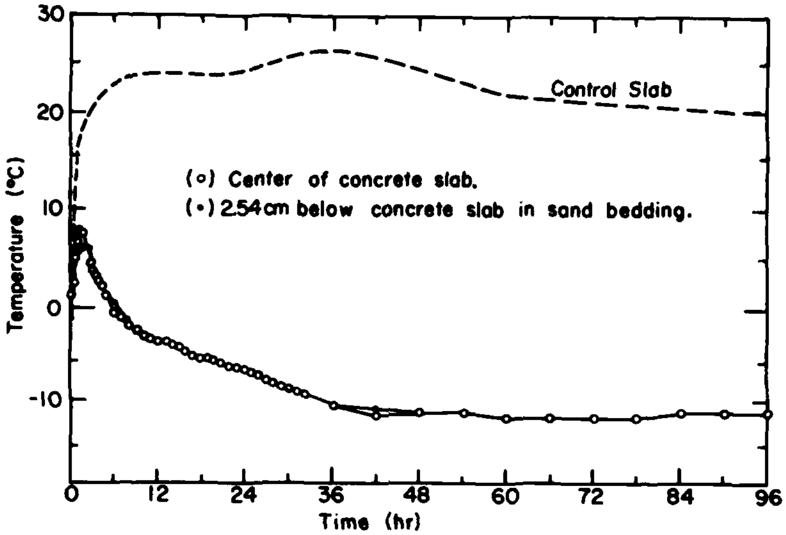

Figure 2. Temperature vs. reg-set concrete age for a 7.5-cm-thick slab and sand bedding exposed to -9.5°C temperature [after Houston and Hoff (1975)].

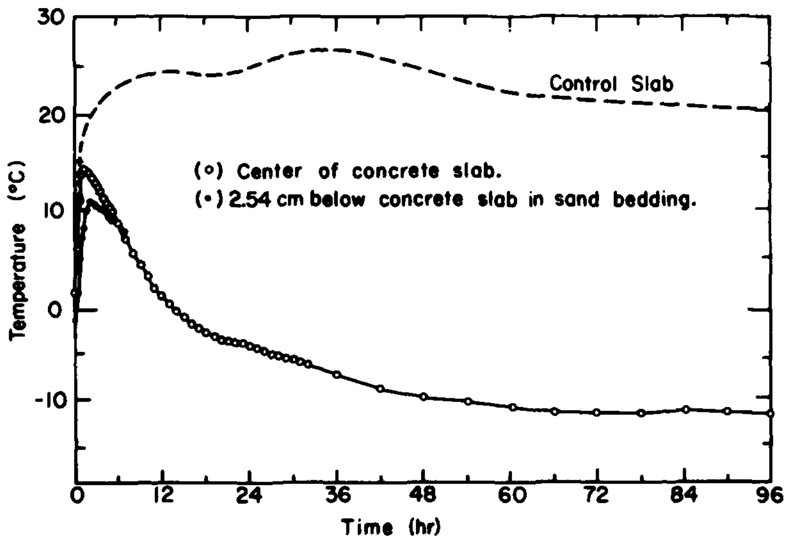

Figure 3. Temperature vs. reg-set concrete age for 15.2-cm-thick slab and sand bedding exposed to -9.5°C temperature [after Houston and Hoff (1975)].

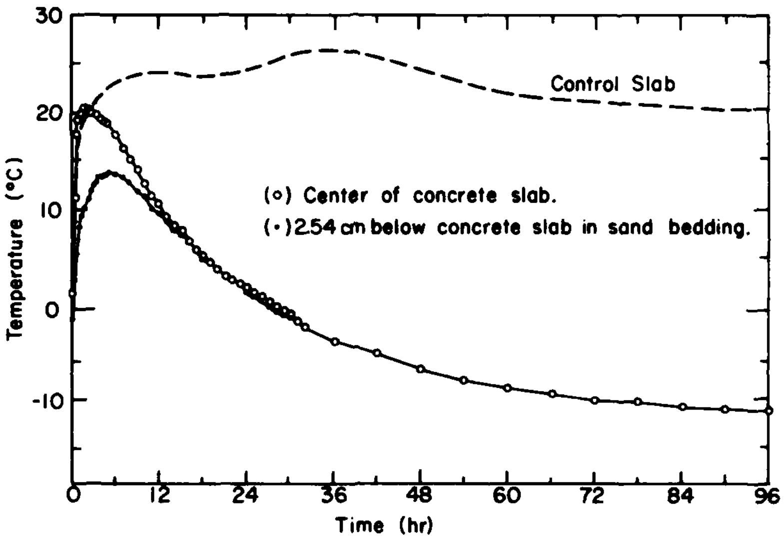

Figure 4. Temperature vs. reg-set concrete age for a 30.5-cm-thick slab and sand bedding exposed to -9.5°C temperature [after Houston and Hoff (1975)].

7.5 cm thick slab in 2.3 hours. These temperatures could melt frozen soils beneath slabs depending upon the initial temperature of the soil and its ice content.

The results of the compressive strength tests performed on the cubes cut from the slabs are shown in Figure 5. The lines in this figure show that compressive strengths at -9.5°C were greater for the thicker slabs. The strengths of the specimens from the bottom half of the 30.5 cm thick slab exposed at -9.5°C were slightly greater than those from the 15.2-cm-thick slab cured at 21°C; however, strengths of specimens from the top half of the slab were lower than those cured at 21°C for 28 days. These observations are generally consistent with the concept that the longer the initial temperature can be maintained above freezing the greater the strength of the slab.

The bottom surface of the 30.5 cm thick slab was protected from the constant -9.5°C air temperature by the sand base as indicated by the temperature rise in the sand at a depth of 2.54 cm (see Fig. 4). Also, it is reasonable to assume that the bottom portions of all the slabs on sand bases would have greater strengths than the top portions. The greater strengths of the bottom half of the 30.5 cm thick slab over the slab cured at 21°C can be explained by the supposition that nearly full strengths were developed in both cases and the difference in strength lies in the accuracy of the strength measurements.

Line	Slab thickness (cm)	Storage temp. (°C)
A	30.5 (bottom)	− 9.5
B	15.2	21.0
C	30.5 (top)	− 9.5
D	15.2	− 9.5
E	7.6	− 9.5

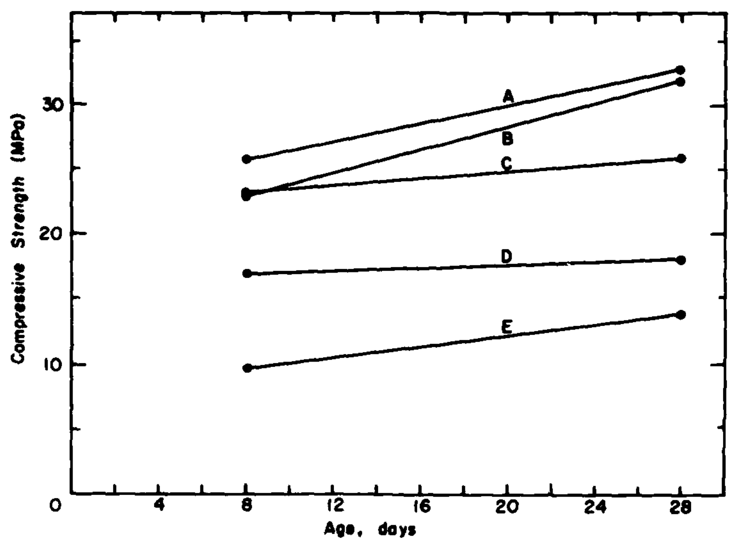

Figure 5. Effect of slab thickness on compressive strength on cubes [after Houston and Hoff (1975)].

FIELD TEST

The results of the laboratory studies on regulated-set cement mortar and concrete at subfreezing temperatures were encouraging enough to justify conducting a controlled field test to verify the laboratory studies and to identify problems in placing regulated-set concrete under field conditions at below freezing temperatures. The details of these tests are being reported by Houston and Hoff (1978), so only a brief summary of results is presented here.

Two 3.7 m by 3.7 m by 20 cm-thick test slabs were poured out-of-doors in Hanover, New Hampshire, during January, when the ambient air temperature was approximately -9.5°C. Figure 6 shows the forms just before placement of the concrete. The bottom of the forms was covered with a polyethylene plastic sheet. The two rows of cylinders seen near the centers of the slabs are plastic forms ("push-outs"), used to facilitate obtaining compressive strength specimens. By leaving the specimens in these molds (in the slabs), the temperature and other conditions of the specimens were approximately the same as those of the slabs during

Figure 6. Forms for reg-set concrete test slabs, Hanover, NH.

curing. These slabs were founded on 15 cm of clean sand over a gravel subgrade. Temperatures were measured at various elevations within the concrete slabs and beneath them to a depth of 30 cm.

The concrete mixture used in these slabs was essentially the same as that used in the laboratory study as previously described. This mixture was designed to have a strength of 20.7 MPa at three days under laboratory conditions. The fine- and coarse-aggregate used in the laboratory study was limestone, whereas the aggregate used in these slabs was a siliceous material (trap rock). However, the gradation of the limestone and trap rock aggregates met the same specifications.

The primary differences between the mixtures for the two test slabs were the temperature of the water and the amount of air entrainment. In slab 1, the water temperature before mixing was 12°C producing a concrete temperature of 0°C at the time of placement. The water added to the concrete in slab 2 was heated to 41°C before mixing and resulted in a concrete temperature of 9.4°C at the time of placement.

Both slabs were cured under outside ambient air temperatures with only a polyethylene plastic sheet placed directly on the surface of the concrete to reduce evaporation and sublimation. No temperature protection was provided. A record of the air temperature and the temperature

at the midpoint of each slab as illustrated in Figures 7 and 8 shows the initial rises in temperature in the slabs to be 3.0°C in 3 hours for slab 1 and to be 10.3°C in 1 hour for slab 2.

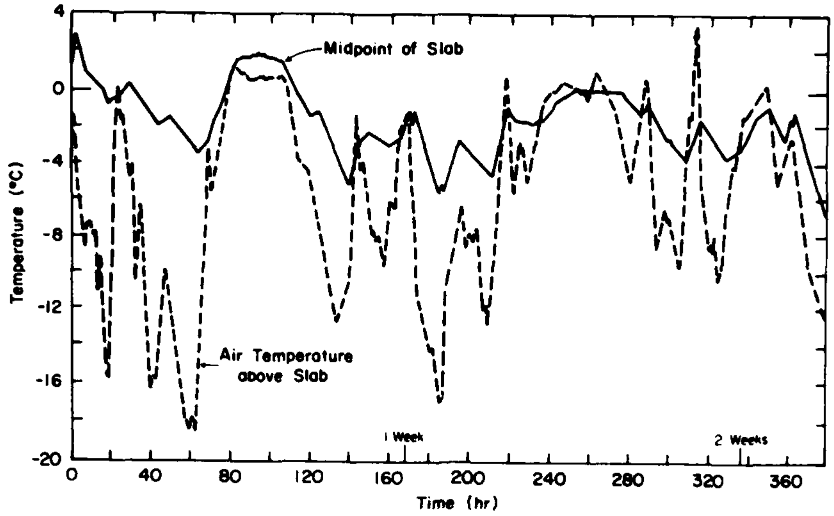

Figure 7. Temperature record for reg-set concrete test slab no. 1 Hanover, NH.

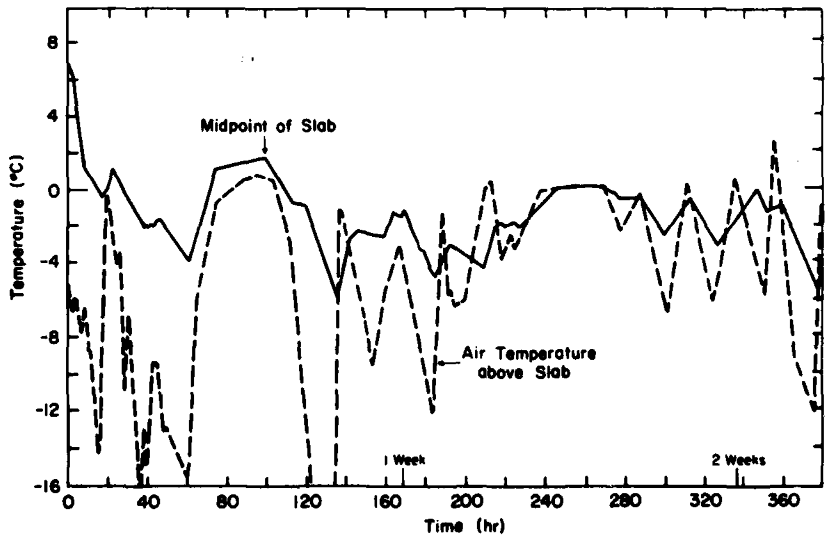

Figure 8. Temperature record for reg-set concrete test slab no. 2 Hanover, NH.

The results of compression tests performed on "push out" cylinders from the two slabs show that slab 2, which was mixed with heated water, gained strength almost twice as fast as slab 1, which was mixed with cold water (see Fig. 9). Comparing the test results for slabs 1 and 2 in Figure 9 with those from the laboratory tests on concrete cylinders shown in Figure 1, the 7- and 28-day strengths for slab 1 are only slightly less than those for the laboratory cylinders placed in -9.5°C temperature immediately after casting (Curve E, Fig. 1) and strengths for slab 2 are slightly greater than those for the laboratory cylinders that were cured at +21°C for 24 hours before exposure to -9.5°C temperature (Curve C of Fig. 1).

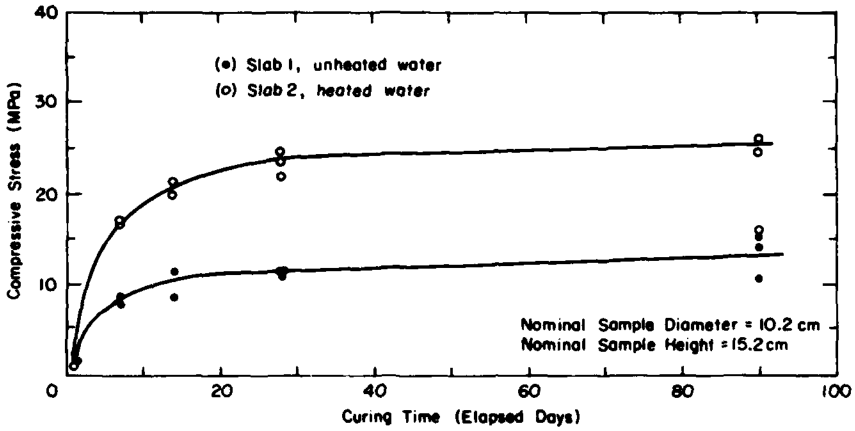

Figure 9. Compressive strength vs. curing time for reg-set concrete "push out" specimens cured in the test slabs in Hanover, NH.

The compressive strengths for the 15.2-cm-thick laboratory slab cured at -9.5°C (Curve D, Fig. 5) are less than those for slab 2 (20-cm-thick) at the 7- or 8- and 28-day ages. The lower strengths of the laboratory specimens may be attributed to the smaller mass of the 15.2-cm-thick slab. The larger mass of the 20-cm-thick field slab provided enough additional heat during hydration so that greater strengths could be developed before the chemical reaction of the cement and water was retarded by freezing.

The strength curves in Figure 9 clearly show the advantage of heating the water for concrete to attain greater strengths. However, if lower strength concrete can meet the requirements for a given structure and heated water is either not available or available only at great expense, it may be acceptable to use cold water with reg-set concrete and perhaps use larger dimensions for the structural members to offset the reduced strength.

FLOOR SLAB FOR AN UNHEATED WAREHOUSE

A. Description

To further develop the methods for placing regulated-set concrete and to make a comparison between normal or type II concrete and reg-set concrete placed in cold weather, a 20-cm-thick unreinforced concrete slab for an unheated warehouse was constructed by a regular construction crew using conventional concrete placing equipment. About one-third of the slab section (called slab "B") was constructed using regulated-set cement and the remaining two thirds (called slab "A") were of the normal type II cement (see Fig. 10 for plan). The foundation for the slab consisted of gravel fill overlaid by about a 15.2-cm-thick layer of dry sand.

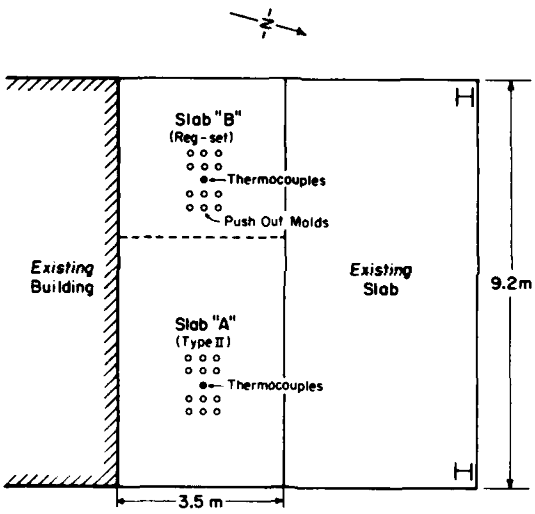

Figure 10. Plan floor slab for warehouse.

To prevent snow and debris from filling the forms before concrete placement, a temporary shelter consisting of a plastic film cover over a wooden frame was constructed over the concrete forms. The film cover was removed from the shelter during the placement of concrete.

B. Concrete Mixtures

The concrete mixtures for both the normal type II and the regulated-set concretes were the same except for the types of cement and the amounts of air entrainment. Table I gives the proportions of the mixture used.

The air entraining agent was excluded from the regulated-set concrete at the batching plant by mistake. The absence of this agent accounts for the lower slump of the reg-set concrete shown in Table I.

Table I. Quantities for one cubic meter

Cement (Type II or Reg-set)	297 kg
Sand	766 kg
Coarse aggregate 3/4 in. max.	1180 kg
Water	158 kg
Air-entraining agent for:	
Normal Type II Cement	309 cm^3
Regulated-set Cement	0
Water-cement ratio	9.53
Slump:	
Normal Type II Cement	11.4 cm
Reg-set Cement	10.2 cm

The fine- and coarse-aggregate were siliceous materials (trap rock). The specific gravity, absorption, and gradation of the aggregate are given in Table II.

Table II.

	Coarse aggregate	Fine aggregate
Specific gravity	2.90	2.71
Absorption, %	0.6	0.8
Sieve size, cumulative		
Passing, %		
3/4 in.	98	100
1/2 in.	54	100
3/8 in.	25	100
No. 4	5	100
No. 8	3	87
No. 16	0	63
No. 30	0	36
No. 50	0	15
No. 100	0	7
No. 200	0	6

Before mixing, the temperature of the aggregate was 3.9°C and the heated water was 54°C, which resulted in the temperature of the final mixtures, at the time of placement, of 17.2°C and 16.1°C for the Type II and reg-set concretes respectively. Ambient air temperatures ranged from -11°C to -8°C during concrete placement.

The concrete for both slabs was mixed in a truck-mounted concrete mixer. The ingredients of the type II concrete were loaded into the mixer in the normal sequence; i.e., the aggregate and cement were placed in the mixer dry at the central batching plant and the mixing water was added at the construction site just before mixing and placing the concrete. In the loading sequence for the reg-set concrete, however, the aggregates were loaded in the mixer at the batching plant; at the site water was added to the mixer, and finally the reg-set cement was added. Mixing was completed in about 5 minutes (at a mixer drum speed of 11 RPM) after the last ingredient was added to the mix. The placement was by chute directly into the forms (see Figs. 11 and 12). The top surfaces of both slabs were finished by hand using normal screeding and troweling procedures.

C. Curing

The top surface of the regulated-set concrete slab was protected from water evaporation and sublimation by placing a 0.15-mm-thick plastic film of polyethylene directly on its surface. No protection from freezing was provided. The type II concrete slab was protected from freezing by an enclosure consisting of a wooden frame covered with plastic film. The enclosure was heated with gas burners for 24 hours (see Fig. 13) and then the surface of the concrete was covered with a

Figure 11. Placing reg-set concrete in floor slab "B" for a warehouse.

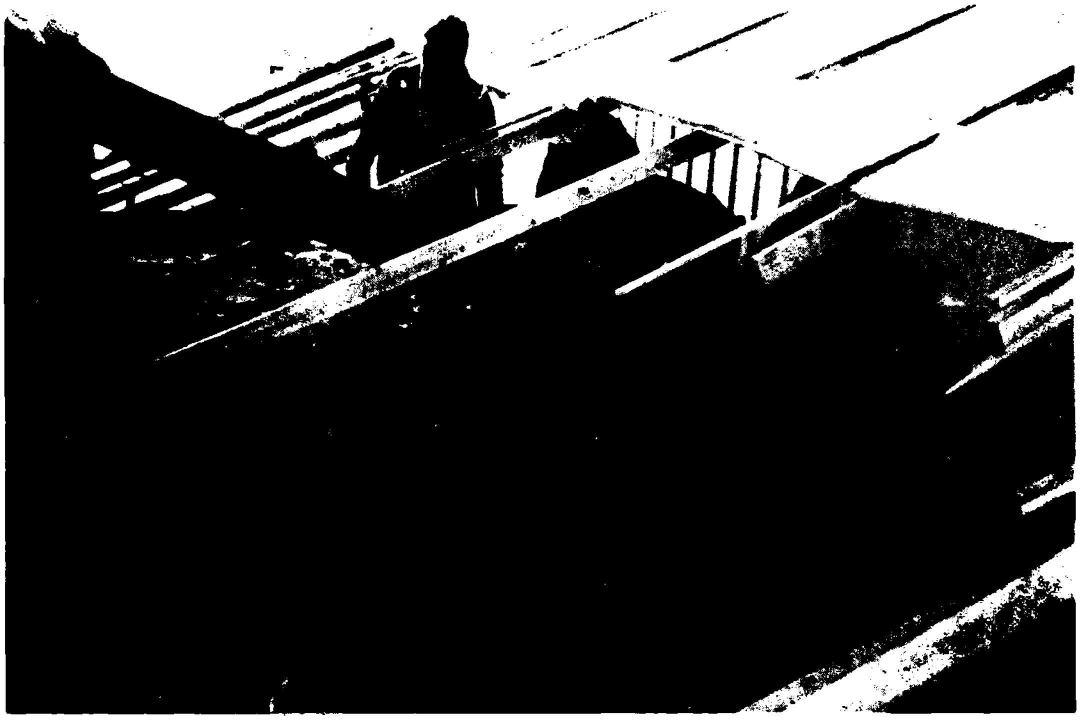

Figure 12. Placeing type II concrete in floor slab "A" for a warehouse.

0.61-m-thick layer of loose hay. The enclosure was left in place to protect the hay from the weather and from being blown away (see Fig. 14). The hay and enclosure were left in place for two weeks after concrete placement.

It was interesting to observe that within 30 minutes after the reg-set concrete was poured the workmen were standing on it to erect the enclosure to protect and to heat the adjacent type II concrete slab.

D. Temperature

Temperatures were measured periodically by means of thermocouples within and immediately beneath the regulated-set and the type II concrete slabs. The thermocouples were positioned along a vertical axis near the midpoint of each slab (See Fig. 10). In addition to the thermocouples, electrical resistance-type temperature sensors were placed also at the midpoint of each slab and in the air just above the slabs. Except for brief periods when the recorder malfunctioned, the temperatures at these locations were recorded continuously. The records of the temperatures at the midpoints of the slabs and the ambient air temperatures are shown in Figure 15.

Figure 13. Replacing plastic film of protective shelter and installing gas heaters to pretect type II concrete floor slab for a warehouse.

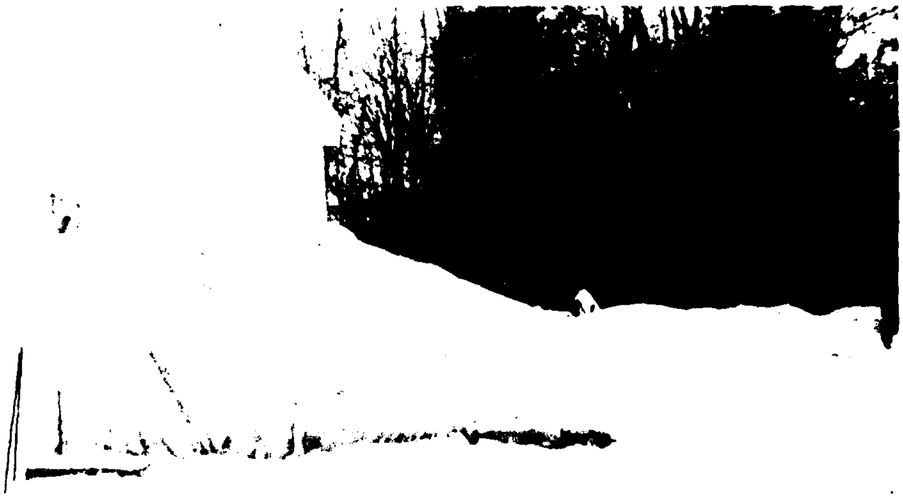

Figure 14. Completed protective shelter over the type II concrete floor slab for a warehouse.

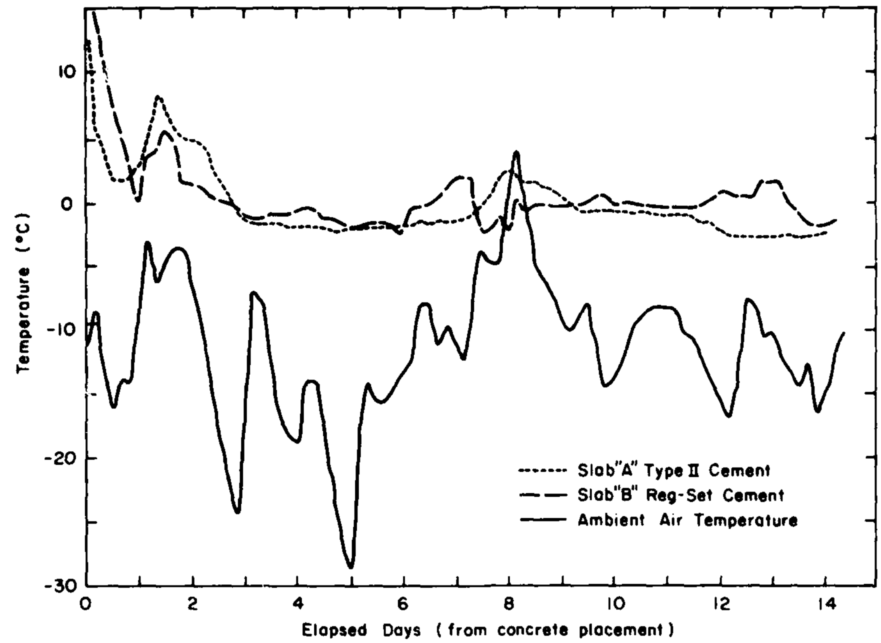

Figure 15. Fourteen-day temperature record of the concrete floor slab for the warehouse.

Figure 16 shows the temperature of the midpoints of the slabs for the first 24 hours. The temperatures of the concrete in both slabs immediately after placement was about 17°C. In comparing the temperature changes with time, it is clear from Figure 16 that the regulated-set concrete remains at a higher temperature after placement than the type II concrete despite the fact that the regulated-set slab was exposed to the lower outside air temperatures and the air above the type II slab was heated to maintain the temperature within its enclosure above -5°C. Tests for the time of setting of concrete mixtures by penetration resistance showed that the initial set of the regulated-set concrete placed at 17°C and cured at -2.5°C occurred about 35 minutes after mixing and the Type II concrete set in about 4 hours. Clearly the reg-set concrete had attained its initial-set before it froze and the type II concrete would undoubtedly freeze if it had not had protection by heating.

The time vs temperature curves in Figure 17 show that the type II concrete slab had nearly a uniform temperature throughout its thickness although the top was slightly warmer than the bottom. The curves for the reg-set slab in Figure 18 show that this slab had a slightly larger temperature gradient, but the top of the slab was colder than the bottom because the top was exposed to ambient air temperatures below -7°C while the bottom benefited from the warmer temperatures below -7°C.

The higher temperatures at the bottom of the regulated-set concrete slab are reflected by the higher temperatures in the sand base beneath the slab. In the sand at a point 2.5 cm below the type II concrete slab base, the temperature increased from -8°C to just above 0°C in about 6 hours; however, during the same time interval and at the same

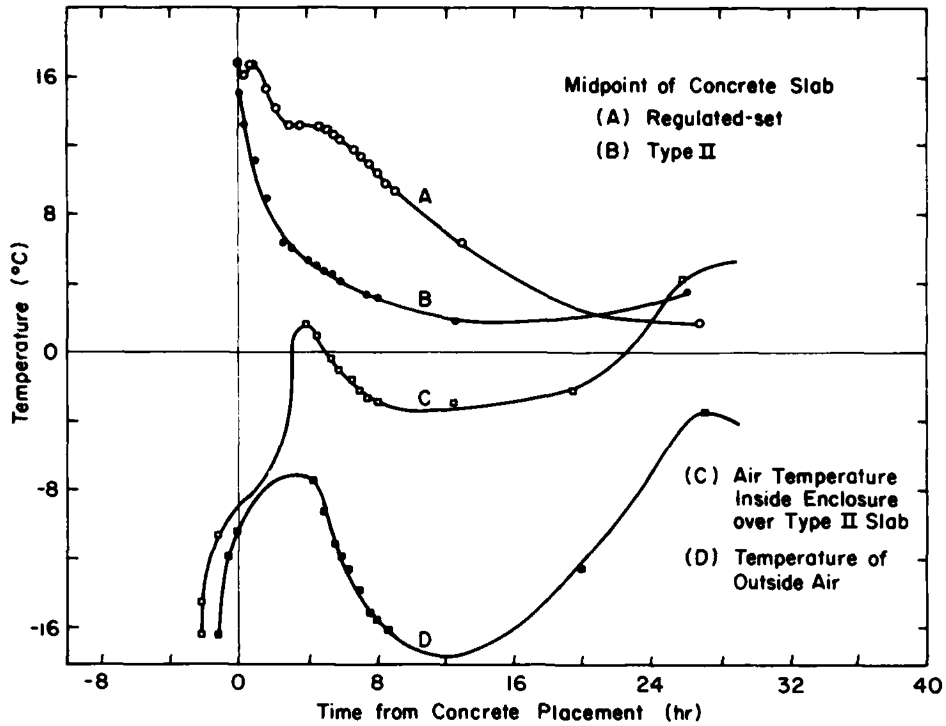

Figure 16. First 24 hours of temperature record for the concrete floor slab for the warehouse.

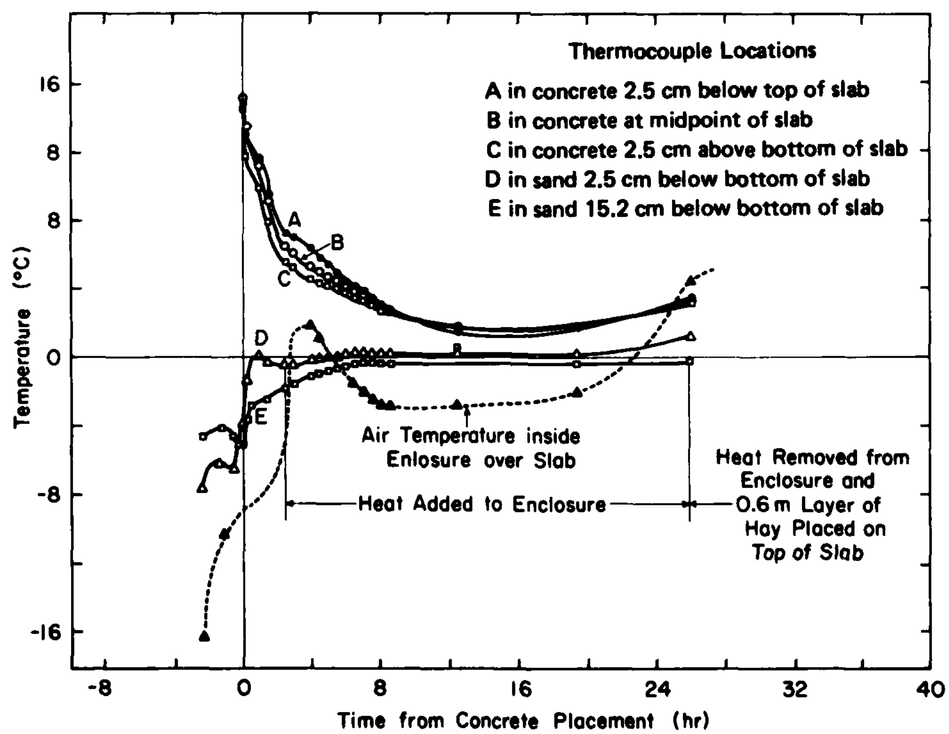

Figure 17. Warehouse type II concrete slab temperature record at different depths.

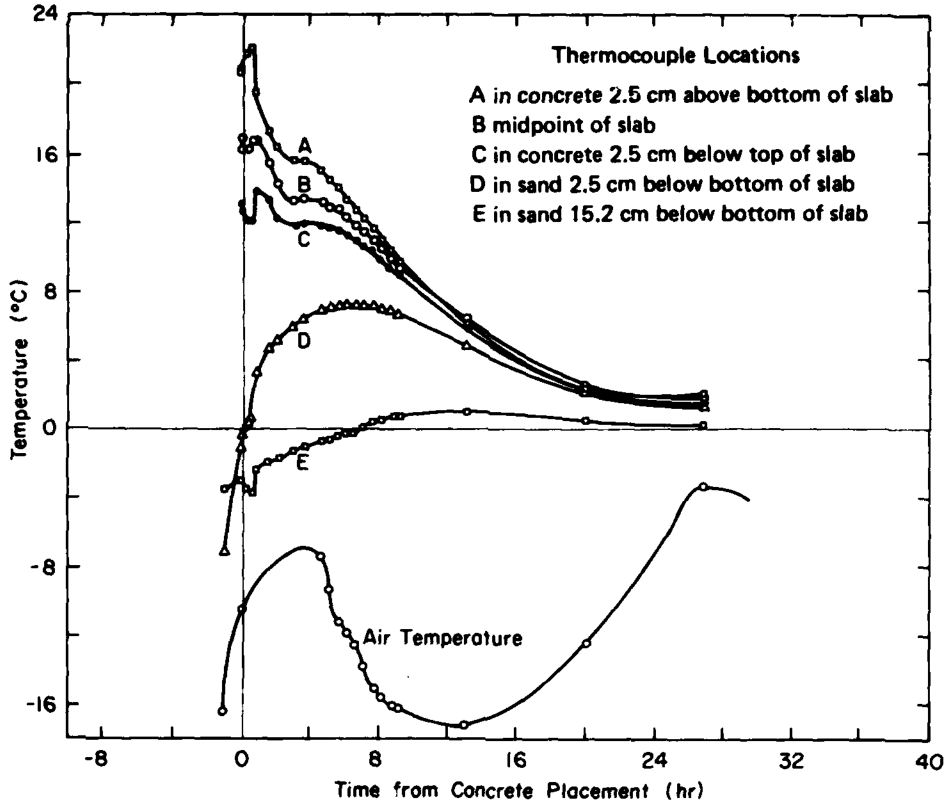

Figure 18. Warehouse reg-set concrete slab temperature record at different depths.

depth beneath the regulated-set slab, the temperature rose from -16°C to +8°C. At 15.2 cm below the reg-set slab the temperature rose to +1°C, while the temperature at the same depth below the type II slab attained a maximum of -0.4°C during a period of 10 hours. In general, the temperature curves indicate that the entire depth of both slabs was above freezing long enough to gain enough strength to resist damage due to freezing.

E. Concrete Test Samples

Three different types of cylindrical samples were taken from the concrete slabs for compressive strength tests at different ages. At the time of concrete placement, six samples, 15.3 cm diameter by 30.5 cm long, were cast using the concrete that was placed in each slab. The samples cast from the type II slab were used as control samples and after casting at -9.5°C they were cured at +21°C in a standard humid room that was maintained near 100 percent relative humidity. Four of the six samples cast from the regulated-set concrete were cured out of doors adjacent to the regulated-set concrete slab; i.e., they were cured under conditions similar to those of the reg-set slab. The remaining two samples were cured out of doors for 19 days and then in a standard humid room for 14 days before being tested for their 28-day strength.

A second group of samples, 10.2 cm in diameter by 20.3 cm long, were poured into plastic molds, "push out molds," that were embedded in the slabs (See Figs. 11 and 12). Samples formed in this manner were cured in the slabs where the heat of hydration from the slab mass maintained the temperature of the concrete above freezing long enough for it to gain sufficient strength to resist damage from eventual freezing.

An examination of the "push out" samples from the regulated-set concrete slab revealed that many of the samples contained air voids along their cylindrical surfaces thus making them unsuitable as representative samples for testing. Therefore, six samples were cored from the regulated-set concrete slab with a diamond core bit using water as a drilling fluid. Three of these samples were tested immediately after coring to determine the 14-day strength and the remaining three were returned to their holes in sealed plastic bags for use in the 18-day compressive strength determinations.

F. Compressive Strength

Unconfined compressive strength tests were performed on the concrete samples after curing for periods of 1, 3, 7, 14, and 28 days. The results of these tests are shown in Figure 19. The curves shown on these figures portray a reasonable representation of the limited amount of data.

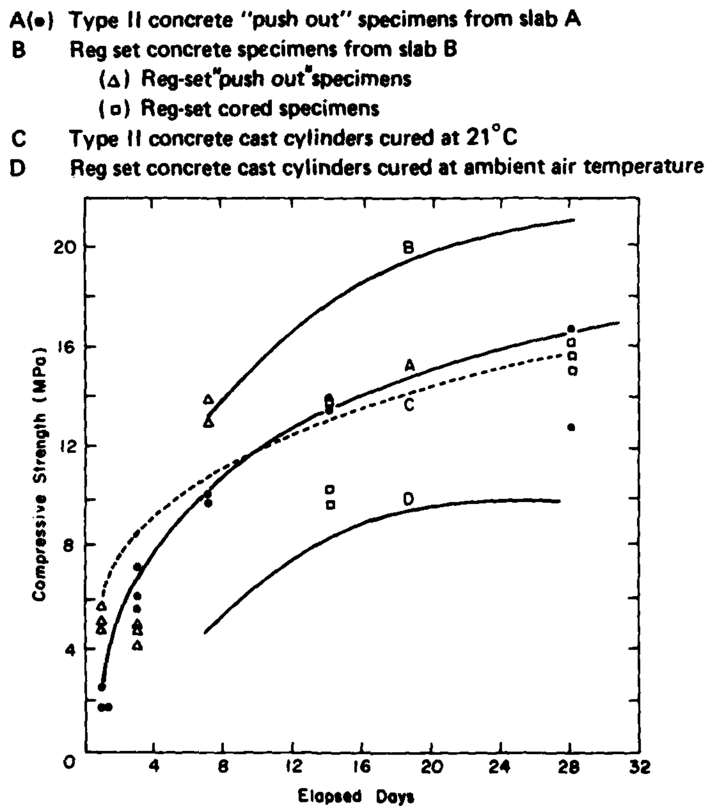

Figure 19. Compressive strength vs. concrete age for concrete in floor slab of the warehouse.

The cast cylinders of Type II concrete that were cured under standard conditions are considered the control strength test samples. Figure 19 shows that the average strength of the control samples is about twice that of the regulated-set cast cylinders and 25% greater than the "Pushout" and cored samples cured in the slab. The low strength of the cast cylinder of regulated-set concrete can be explained by the fact that the small mass of the samples did not provide enough heat during hydration to prevent them from freezing before they attained sufficient strength to resist damage from freezing.

Concrete cylinders cured in the two slabs had nearly the same strength after 28 days of curing, although the regulated-set concrete had a much higher 24-hour strength than the Type II concrete.

The limited data from this small project indicate that reg-set concrete placed in ambient air temperatures as low as -9.5°C without external protection against freezing can attain about the same strength as Type II concrete that is protected from cold weather by shelter, heat, and insulation.

CONCLUSIONS

A. The limited data available on the use of regulated-set concrete for cold weather construction indicate that it is practical to place this type of concrete on frozen thaw-stable soil in ambient air temperatures as low as -9.5°C without furnishing protection against freezing provided that:

1. The mass of the concrete member is large enough so that its internal heat can sustain the hydration until the concrete is strong enough to resist damage from freezing. (A slab thickness of 20 cm or more can meet this requirement.)

2. The temperature of the concrete mixture at the time of placement is higher than 9.5°C.

3. The sequence of adding the concrete ingredients to the mixer is such that the regulated-set cement is added last, just before mixing.

B. A regular concrete construction crew using conventional concrete mixing and placing equipment can build structures with regulated-set concrete in below-freezing temperatures with little difficulty.

C. The 28-day strengths of the regulated-set concrete of sufficient mass placed in -9.5°C ambient air temperatures and unprotected from freezing are approximately the same as the strengths for normal Type II cement that have been protected from freezing while curing.

D. Although the cost of regulated-set cement may be as much as 20% higher than that of type II cement, the total cost of the reg-set concrete in place, without the expense of building a shelter and heating

it, should be much less, in addition, there is a real savings in energy and material required for heating.

It is felt that regulated-set cement used in concrete and mortar for cold weather construction has the potential for savings in time, energy and materials. However, it is now time for the field construction forces to put this material to the true test by using it in useful concrete structures.

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DESIGN OF THERMOACTIVE FORMS AND POURING CONCRETE STRUCTURES IN THERMOACTIVE FORMS IN CONDITIONS OF THE NORTHERN ZONE *

By V.D. Topchii[1]

INTRODUCTION

The use of thermoactive forms for poured concrete construction in conditions of the northern zone is efficient both with respect to economic considerations and with respect to the use of technology. It is now possible to deliver low temperature concrete mixtures to the work site and thus reduce heat losses during transportation. Thermoactive form panels may be used for preheating the ground or warming "old" concrete of the structure. Optimum temperature and moisture conditions may be created inside the form. The reliability of monitoring the curing processes of concrete structures and the possibility of providing local heating to parts of the structure located in the most unfavorable conditions constitute the exceptional advantages of this method.

Research which developed initial data for designing and constructing thermoactive forms (used in outside air temperatures of -35 to -45°C) was carried out at TsNIIOMTP.

The specific capacity of the heaters of the thermoactive form panels may be defined as the sum of the useful powers, that is, of the power which can be expended for heating the concrete and compensating for external heat losses. The useful power can not be assumed to be arbitrarily great and depends on the permissible temperature drops between the core and the peripheral regions of the structures. The useful power was chosen in accordance with the calculated rate of heating the layers of concrete adjacent to the form and depends on the ability of the concrete to conduct heat inwardly, which is connected with the thermophysical characteristics of the concrete at different ages (for example, with the variation in the thermoheat conductivity of the concrete mixture and the concrete).

In addition to the technological limitations there are limitations predicated by economic considerations. For monolithic structures (including thin walled structures) it is not efficient to use a rigid heat treatment at high temperature since the heat losses are increased. Furthermore, it is not possible to obtain a noticeable effect from shortening the form turnover time, as takes place in factory conditions in the casette production of prefabricated structures. In practice, the practical form turnover time is within the limits of 5-10 days.

HEAT TRANSFER COEFFICIENTS: RELATION TO ENVIRONMENT AND STRUCTURAL GEOMETRY

The heat losses depend on the magnitude of the temperature drops

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*Not presented at seminar.

between the panel framework and the outside air and the magnitude of the heat transfer of the insulation.

The insulating properties of the form depend on its moisture and ventilation. The overall heat losses of a thermoactive form may vary within a wide range depending on the force and direction of the wind, the initial moisture of the insulation, the nature of the mass transfer in the insulation during the operation of the heaters, and the number of whole bridges (which are open metal parts of the form: ribs, fasteners, braces and so forth). A determination of the heat transfer coefficient of a form in the case of variation of the above mentioned conditions by means of calculation is difficult. For practical purposes we determined a number of relationships for the heat transfer coefficient on a special test stand.

Cold bridges (Table I) exert a significant influence on the amount of heat losses. Covering cold bridges with a layer of slag wool 10-15 mm thick, or protecting them with plywood, reduces heat loss by 25-70%.

Choosing the specific capacity of thermoactive form heaters is both a technological and economic task. The technological aspects are connected with the nature of the chemical and physical processes in the hardening concrete, in the case of an external heat flow and the heat stressed state of the structure. The economic aspect of the problem involves an evaluation of the heat losses and the expediency of changing the periods of heat treatment in one direction or another.

It is not difficult to see that in the case of low specific capacities of the heaters, and with a specific ratio of the indicators of the insulation properties of the form, and the external temperature, the process of raising the temperature may continue for a fairly long time. In other words, the rate of raising the temperature is low. The maximum heating temperatures also are low. The rate of the temperature rise and the maximum temperature in the form increase with an increase in the specific capacity with all other conditions being equal. These remarks are correct in the case of a 1-dimensional system and in the case of heating a fairly massive structure. In structures with a fairly high modulus of the surface covered by forms the temperature drops between the periphery and the core, as a rule, are not great. The rate of the temperature rise in the form, on the other hand, is significantly higher than in the case of heating massive structures. The maximum heating temperatures do not depend on the mass of the concrete to be heated.

Upon achieving the maximum temperature, the process of heat transfer to the concrete slows down significantly and finally approaches zero. The amount of heat entering the structure is determined by the value of the heat transfer coefficient of the concrete and the temperature drops between the periphery and the core. The heat flux drops as the temperatures are equalized and all power expended becomes lost.

Since the magnitude of the losses depends on the temperature drop and the insulating properties of the form, it is possible to change the maximum permissible temperature in the form and the rate of the temperature

rise not only by means of changing the specific capacity of the panel heaters, but also by means of structural changes in the insulation, depending on the temperature of the outside air.

Experience in using different types of thermoactive forms has made it possible to establish that the maximum value of the heat losses should not exceed 35% for a reuseable form, and 20-25% for unit forms, large panels, and form coils.

The recommendations contained in Table 2 may be used in the stage of preparation for winter concrete work in planning energy and material resources.

EXPERIMENTAL RESULTS OF THERMAL HARDENING

Heat hardening of concrete structures is a technological problem. The difficulties are connected with the phase conversions taking place during the hydration of the cement, with the changes in the structure of the concrete as a solid (as the hydration process proceeds and under the influence of the internal mass transfer and mass exchange with the environment), with the variation in time of the physical-mechanical characteristics (strength, modulus of elasticity, coefficient of heat expansion) and their dependence on the heat treatment, with the variation in time and temperature of the thermophysical characteristics (heat conductivity and heat capacity).

The boundary conditions may be obtained by an experimental calculation accounting for the heat stressed state of the concrete and its strength and deformational properties. Naturally, the solution of the latter problem also is connected with the configuration of the cross-section of the structure.

TsNIIOMTP has proposed a model of the heating process, using experimental data on the rate of heat liberation of the hydration reactions and the variation in heat conductivity in the case of external heating.

In general form, the heat conductivity equation for the 1-dimensional problem (heating a wall of thickness "B") has the form:

$$c_v \frac{\partial t}{\partial \tau} = \frac{\partial}{\partial x} (\lambda \frac{\partial t}{\partial \tau}) - \frac{\partial q}{\partial \tau} . \qquad 1)$$

Therefore, the boundary conditions may be written in the case of 2-sided heating as:

$$t/\tau = T_o = t \qquad (2)$$

and

$$\frac{dt}{dx}\Big|_{x=b/2} = 0 , \qquad (3)$$

where: B = total thickness of the structural element
c_v = the heat conductivity of a concrete structural element
t = time
τ = elemental element thickness
x = the distance from form heat source to any structural boundary
q = heat of hydration of the concrete
T_0 = temperature variation over some time interval.

The boundary conditions $x = 0$ may be written with the use of the characteristics of the thermo-active form. The capacity of the heaters N and the thermal resistance of the insulation in the form R may thus be stated as:

$$\left(-\lambda \frac{\partial t}{\partial x} + \frac{t-t_o}{R}\right)\Big|_{x=0} = N(\tau), \tag{4}$$

where λ is the heat conductivity of the concrete member.

In the case of controlling the heating conditions according to the temperature reading on the form, instead of condition (4), it is possible to write the function of the temperature variation in time, hence:

$$t\Big|_{x=0} = T(\tau) \; . \tag{5}$$

As a rule, monitoring and measuring instruments provide for linear measurements of temperature, thus function (5) is sufficient for solving the equation. Since two terms in equation (1) (λ and c_v) vary in time it is proposed they may be written in the form below:

$$\lambda = \lambda_o(t) - K_1(x,\tau)[\lambda_o(t) - \lambda_\infty(t)], \tag{6}$$

and

$$c_v = c_o(t) - K_2(x,\tau)[c_o(t) - c_\infty(t)], \tag{7}$$

where (λ_o, λ_∞) and (c_o, c_∞) are the values of the heat conductivity and the heat capacity at the beginning and end of the process. In the range of variation of the heat treatment temperatures it is possible to ignore the relationships between λ_o, λ_∞ and c_o, c_∞ and the temperature. In this case, equations (6) and (7) will acquire a simpler form:

$$\lambda = \lambda_o - K_1(x,\tau)(\lambda_o - \lambda_\infty), \tag{8}$$

and

$$c_v = c_o - K_2(x,\tau)(c_o - c_\infty), \tag{9}$$

where the functions $K_1(x,\tau)$ and $K_2(x,\tau)$ vary from zero to unity as the hydration reaction proceeds. Thus, it is rational to express these terms as functions of heat, which will make it possible to use the experimental data:

$$K_1(x,\tau) = \frac{1}{q_\infty} q(x,\tau), \tag{10}$$

and

$$K_2(x,\tau) = \frac{1}{q_\infty} q(x,\tau), \tag{11}$$

where q and q_∞ are the hydration heat in any segment of time and the total hydration heat, respectively.

After substituting the values λ and c_v into equation (1) we obtain the expression:

$$[c_o - \frac{q}{q_\infty} (c_o - c_\infty)] \frac{\partial t}{\partial \tau} = \frac{\partial}{\partial x} \{[\lambda - \frac{q}{q_\infty} (\lambda - \lambda_o)] \frac{\partial t}{\partial x}\} + \frac{\partial q}{\partial \tau} . \tag{12}$$

In this form it is easy to solve equation (12) with the known variations of λ, c and q in time. The approximation method may be used for engineering calculation with the aid of computers. The essence of this method comes down to the fact that the initial (and perhaps, any intermediate) value of λ and c_v is substituted in the equation, and the heat of hydration is ignored initially. After determining the temperature-time function for the characteristic points of the cross-section of the structure, the equation is solved again using the functions λ, c_v, and $q(\tau)$, obtained experimentally for the corresponding temperatures. Thus, the temperature-time functions usually are determined more precisely, with one or two trial solutions. For larger cross-sections (in structures with a surface modulus of 2), it is necessary to calculate three solutions.

A large series of experiments was conducted in order to obtain the relationship between the heat transfer coefficient and the temperature-time factor. The variables were: the rate of a temperature rise, the maximum isothermal aging temperature, and the preliminary aging time. The water-cement ratio and the initial water content of the mixture, and in certain experiments, the external mass exchange conditions, varied. The method of determining the thermophysical characteristics was based on temperature measurements at specific points on the concrete samples in quasi-stationary conditions. The concrete samples were aged in thermoactive forms measuring 350 x 250 mm and were 350, 700, and 1000 mm in height.

SUMMARY AND CONCLUSIONS

In our experiments the initial values of the heat exchange coefficients correspond to the data acquired previously in the Teploproyekt Institute, but we did not once succeed in observing an increase in heat conductivity after 12-24 hours.

In the absence of external mass exchange, heat conductivity during the first 12-14 hours decreases more intensively in the case of high heat treatment temperatures and with a reduction in the initial water content. This may be explained by the general reduction of the water content in connection with the hydration processes. The formation of crystalline structures and the increase of the heat conductivity of the solid phases cannot compensate for the reduction in the heat transfer of the liquid phase.

The experiments showed that the mass exchange with the external environment in practice has an effect on the variation in heat conductivity only in layers 100-150 mm thick, directly to open surfaces. Insulation of these surfaces with coverings of polyethylene, rubber, or parchment makes it possible to consider that the heat conductivity in all cross-sections along the vertical is identical in the case of identical temperature-time factors.

For measuring the hydration heat the author developed a special calorimeter, the distinguishing features of which were: a thermoactive form for a weighed amount of a cement-sand solution in a Dewar vessel, a heat absorbing element, and a system for monitoring temperatures and taking account of the external heat flow. Heat losses from the Dewar vessel were excluded thanks to the fact that the temperature of the outer water sleeve was maintained strictly in accordance with the temperature of the cement-sand solution. The results obtained for Portland cement from the Voskresenskiy Plant diverge markedly from the results calculated according to Teploproyekt graphs.

A special test stand was used for determining the temperature deformations and stresses and also the strength of concretes of different compositions subjected to nonstationary heat treatment conditions lasting from 2 to 36 hours. These results made it possible to make a quantitative evaluation of the permissible heat stressed states of concrete and reinforced concrete structures with external heating.

The optimum heat treatment condition in a thermoactive form were calculated for the most characteristic types of industrial and civil structures.

Table I

Relationship between the Heat Transfer Coefficient of Form Panels on the Dimensions of Cold Bridges

Ratio of the Parameters of Frame Ribs to Panel Area	Area of Open Ribs (cm^2)		Heat Transfer Coefficient	
	Surface	Cross Section	$W/(m^2 x °C)$	%
4:1	600	1600	3.4	100
4:1	1200	1600	4	115
4:1	2000	1600	5.02	147
6.5:1	1000	1600	6.26	183
6.5:1	2000	2600	8.3	243
8:1	1200	3200	8.3	243
8:1	2200	3200	9.7	283
8:1	3200	3200	11.6	340

Table II

Recommended Specific Capacity of Thermoactive Form heaters

Calculated Air Temperature °C	Specific Capacity of Heaters, W/m^2, with the modulus of The Form Covered Surface				
	2-4	4-6	6-8	8-10	Above 10
-20	600	700	800	900	1000
-25	650	750	850	950	1100
-30	700	800	900	1000	1100
-35	800	900	1000	1100	1200
-40	1000	1100	1200	1400	1600
-45	1200	1400	1600	1800	2000

SECTION V:
CONSTRUCTION EXCAVATION TECHNIQUES:
APPLICATION TO COLD REGION CONDITIONS

EXCAVATION OF FROZEN MATERIALS

By Harlan E. Moore[1] and Francis H. Sayles[2]

INTRODUCTION

a. Background

Historically, large scale earthwork construction projects were not attempted in Alaska until recent years. A typical construction season of about four months began with the advent of spring breakup, continued at a frenzied pace through the summer and early fall, and terminated with the early first snows of winter. It was this practice that gave rise to the early Alaskan phrase "Termination Dust." As the first snows of winter covered the adjoining mountains, the typical construction worker terminated his stay in Alaska and went "outside" for the winter, to return again in the spring.

With the advent of larger, more expensive equipment, winter work has become more and more prevalent throughout Alaska. To a large extent, this is a normal progression of our industrial development as well as the ever increasing spiral of construction costs. The cost of heavy equipment has been rising steadily and, therefore, a contractor finds he must keep this equipment working almost all year round in order to justify owning it. In the past 10 or 15 years, large, heavy bulldozers with heavy-duty rippers have become available and the contractors are finding that these machines can cope with the rigors of frozen ground and winter excavation. Before this time, machinery with the necessary weight and power was simply not available. In recent years, several large construction projects have been undertaken in Alaska which, by the nature of the work, required scheduling winter work to expedite the efficient movement of large masses of material during the limited season.

Often it is more advantageous to do particular types of work in the winter rather than in the normal summer construction season to take advantage of the strength and imperviousness of frozen soil. For example, ground which is soft and wet in summer is much easier to traverse when frozen. Therefore, in the cold winter months, it is often more economical and expedient to concentrate on the parts of a construction project where the access over soft ground is frozen. Where excavation extends well below the existing water table, it is practical to excavate these areas in the winter by allowing the ground to freeze before excavating and, thereby, minimize the risk of flooding the open excavation during construction. Also, for work being done in or around rivers, the water in the northern streams is much lower in the winter thus making it possible to excavate under dry stable conditions on the banks. The damage to the environment is considerably less when the ground is covered with snow and frozen; therefore, it is often advantageous to utilize the frozen conditions simply to protect the fragile subarctic.

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b. Purpose

The purposes of this report are: to describe some of the techniques used in excavating frozen soils for a flood control project in the Subarctic during the winter; to report some of the problems that are encountered during construction, and to identify the soil parameters that control the difficulty of ripping frozen soils.

c. Scope

This report covers the ripping method used in excavating frozen earth for the major components of the Chena River Lakes Project near Fairbanks, Alaska, and describes the problems encountered during the winter construction period in this subarctic environment.

DESCRIPTION OF PROJECT STUDIED

a. General

The Chena River Lakes Flood Control Project is located near Fairbanks, Alaska, and consists of three major components: (1) the Moose Creek Dam, which is a 11.8-km-long earthfill dam with an average height of 9.1 m, (2) a system of levees along the Tanana and Chena Rivers, and (3) an earthfill dam on the Little Chena River, 1695 m long, with a maximum height of 31.7 m (see Fig. 1 for locations). The construction of the dam on the Little Chena River has been postponed indefinitely and will not be discussed. This report will describe the winter excavation procedures used for portions of the two other components.

b. Moose Creek Dam Foundation Excavation

The undesirable materials in the foundation of the Moose Creek Dam were excavated in the winter in two stages, by two different contractors.

(1) Moose Creek Dam Foundation Excavation Stage I.

This excavation was performed under contract to excavate approximately three quarters of a million cubic meters of silt and sandy silt from the dam foundation and replace it with selected gravel. The excavated area was from 61 to 76 m wide, 1.5 to 4.57 m deep and approximately 11.4 km long. The work covered by this report was performed during the winter of 1973 and 1974. The contractor had the option of excavating either in the winter or summer. Since all of the materials excavated would be relatively impervious, they were placed in a 1.5-m-thick, 305-m-wide, impervious blanket upstream of the dam. These materials were not required to be compact; therefore, the placement of frozen chunks of soil was considered acceptable. These chunks broke down in the subsequent summer months and were compacted by normal weathering.

The contractor carefully considered the options of summer vs. winter excavation. The summer program had the advantage of affording easy excavation of the thawed materials; however, it had the disadvantage of requiring work over large, soft, marshy areas and a high pumping cost where excavation would often be 1.5 m to 3.0 m below the existing water table. It also required that large amounts of construction equipment be employed in the short summer construction season. This would require leasing additional equipment during the summer and letting the contractor's own equipment stand idle in the winter.

After a close examination of existing soil exploration data, the contractor decided that it would be to his advantage to concentrate his winter efforts in the poorly drained areas and the areas where excavation below the water table would be required. Zones where excavation would not go below the anticipated water table were reserved for work during the summer months. The majority of this work was done with heavy-duty, single-tooth rippers mounted on D-9G Caterpillar tractors (385 hp) to break up the frozen materials and large, self-propelled scrapers to remove the frozen materials from the excavated area. The system of allowing the material to freeze ahead of the equipment was utilized in all areas where excavation below the water table was required.

(2) Moose Creek Dam Stage II Foundation and Channel Excavation

This work consisted of a contract to complete the north portion of the Moose Creek Dam. The project involved foundation excavation similar to that previously described plus several kilometers of rather deep channel excavation. The channel ranged from 3.7 to 11 m wide at the base and from 1.5 to 6.1 m deep with a side slope of 1 horizontal to 3 vertical. Approximately 12.9 km of channel were excavated. This work was performed during the winter of 1974 - 1975 under a contract similar to the previous contract and had the same provisions relating to optional winter or summer excavation.

As an interesting sidelight, Williams and Costs (1976) noted that one subcontractor, with eight pieces of equipment, using an old Alaskan expedient, dug a trench and, after laying logs across the top and covering them with 1-mm thick plastic sheeting, parked his equipment in this "warm" storage (see Photo 10). When covered by 0.15 m of snow, this shelter kept the equipment at about -9.5°C, even though the temperature was -40°C outside. As a result, equipment availability was not much different from that of this subcontractor's summertime operation. The only heat supplied was the warmth of the engines at the time of shutdown and the natural heat of the earth.

The contractor on this work took advantage of the experience gained the year before. Many of his key personnel had worked on the previous contract and, therefore, were intimately familiar with the cold weather construction problems. For this reason, he patterned

much of his work after that of the previous contractor, making some allowance for the different types of equipment that he had at the job site. One major difference in this work was that all types of materials, from frozen silts to sands and gravels, were excavated, whereas, in the previous contract, the excavation consisted entirely of frozen silts and silty sand.

The materials in the channel area consisted of silts and fine sands in the upper portions of the excavations, and occasionally relatively clean sands and gravels were encountered near the lower portion of the excavation. These pervious soils at lower elevations compounded the problems of excavation since a frozen soil water barrier (or curtain) was used in this area as a water barrier. In deep excavations below the water table, the head is much higher, and therefore, if the excavator breaks through the frozen curtain, it is quite possible that the excavation will flood rather than freeze and seal itself. When relatively clean sands and gravels, which can carry large amounts of water, are encountered, this problem is much more severe. On several occasions, the channel excavation flooded quite rapidly and the contractor was forced to build dikes and isolate the flooded areas or risk flooding the entire channel area. The contractor performed quite well on the dam foundation excavation; however, he had rather extreme difficulties in the lower portions of the large channel excavation. Because he was not able to get the majority of this channel to grade, **eventually his contract was terminated.**

c. Tanana River Diversion

This consisted of a contract to excavate a 3.2-km-long pilot channel and construct a dike to divert the Tanana River at a point where severe bank erosion was taking place (see Fig. 1). This work was performed during the winter of 1974 and 1975 under a contract awarded in the early fall, this gave the contractor the option of working in the river prior to freeze-up or working from an ice bridge after freeze-up. The contractor elected to do all of this work after freeze-up because he was able to utilize an ice bridge to gain access to the site and to take advantage of the low flow in the river for ease of diversion.

The contractor had no difficulty constructing the ice bridge for this main access road. The bridge was approximately 76 m wide and consisted of two sections approximately 122 and 91 m long. The thickness of ice was about 1.5 m. This bridge was built during extremely cold weather (-50 to -57°C) in late December 1974 and early January 1975. In late January 1975, the construction of another ice bridge along the levee alignment was attempted. Water was pumped on the top of the ice for approximately 6 days with no success even though the initial natural thickness of ice was about 1 m. It appears that the water was lost through cracks in the ice instead of freezing and building up. The temperature ranged between -18 and -23°C with one day as warm as -12°C. These temperatures are considered to be too warm for construction of an ice bridge.

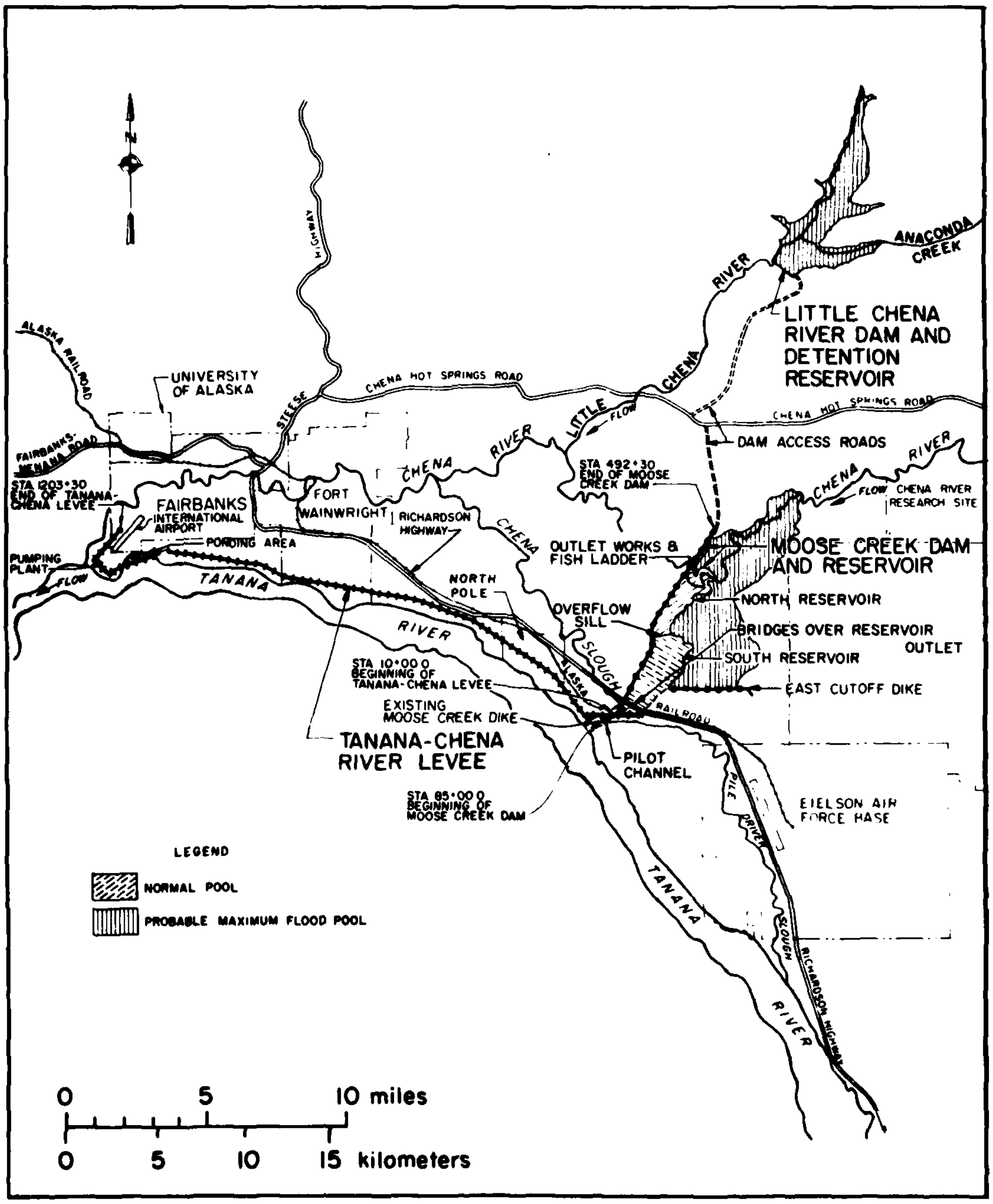

Figure 1. Plan of Chena River Lakes Project, Alaska.

After the ice bridge for the main access road was completed, the excavation of the frozen sands and gravels was extended a few meters below the water table. No attempt was made to allow frost penetration ahead of the excavation below the water table since the contract required placement of large amounts of unfrozen material in the diversion dike. All frozen materials were placed in a large waste berm. The material below the water table was excavated by conventional scraper and pumping methods. Some areas of extremely difficult ripping were encountered on this project. Often, it was necessary to utilize an additional D-9G Caterpillar tractor to force the ripper tooth even a few centimeters into the frozen gravels (see Photo 5). This will be discussed later in the report.

This project was interesting in that, although the majority of the work was done in the coldest part of the winter, considerable work extended into the spring and through the breakup period. Therefore, experience was gained in ripping these materials in the very cold portion of the winter, and in the spring, and early summer.

METHODS OF EXCAVATION

a. Heavy Duty Rippers and Scraper Operations Above the Water Table.

Almost all of the tractors used in the above projects were Caterpillar D-9G bulldozers equipped with single-tooth hydraulic rippers and with shanks approximately 1.2 m long (see Photos 1 and 2). These tractors weigh approximately 31,070 kg, and have a draw bar horsepower rating of 385. At this time, the newer, more modern, D-10 bulldozer was not available, however, a Fiat-Allis, Model 41B was used on an experimental basis by one of the contractors. This tractor weighs approximately 71,690 kg and has a fly wheel horsepower of 524. This machine was used with a hydraulically operated, single-tooth ripper and a shank 1.5 m long. The general opinion of the personnel at the job site was that this machine was not any more efficient than the Caterpillar D-9G model tractors. Even though it was a much heavier and more powerful machine, it appeared that the actual ground pressure on the tracks (and therefore the true ability to pull) was somewhat less than that of the smaller D-9G tractor. If this machine were equipped with the same size ripper tooth as the smaller tractors, it undoubtedly would have been able to perform as well as, or better than the smaller tractors; however, since it is a much larger, more expensive piece of equipment, it would have been a less efficient operation. The scrapers utilized to load the ripped materials were conventional, self-propelled scrapers in the 15-23 m^3 category. The size of the scraper was critical, since moving the frozen soil lumps was similar to handling large boulders. These were well within the capability required (see Photos 3 and 8).

Where exceptionally hard materials were present, the method of ripping was generally that of utilizing a criss-cross pattern with the ripper tooth (see Photo 7). The dozer would operate diagonally across

the area, penetrating as deeply as possible with each pass of the ripper. Depending upon the material being ripped, this depth might vary from less than 3 cm to a meter or more. In general, it would be 2 to 15 cm on the upper hard-frozen crust material, and the penetration would increase with depth in succeeding passes until a final depth of about 1 m was reached (see Photos 4, 5 and 6). In this manner, ripping would progress across an area in parallel lines approximately 1 m apart. After this had been completed in one direction, ripping would begin in a similar operation working at 90° to this pattern diagonally across the excavated area from the opposite side (see Photo 7). By working in this criss-cross pattern, the material could be loosened far more efficiently than by working in straight parallel lines.

Where exceptionally hard ripping was encountered, such as in highly saturated gravels near the surface, where extremely cold temperatures were experienced, one dozer had to be used to pull the single tooth ripper while an additional dozer had to be used to push the tooth. The additional weight of this pushing tractor helped force the point of the tooth down while the additional horizontal force helped to perform the ripping operation. Even with the two-dozer operation, the penetration would be in the order of 5 to 10 cm in the hardest material. In general, after the tooth had penetrated approximately 30 cm in this manner, the ripping became somewhat easier and a single dozer was able to perform the operation. With this extremely difficult ripping, the contractor found that it was necessary to replace the ripper teeth after approximately one hour's work, whereas, on the average, the ripper teeth lasted a 12-hour shift [Kovacs (1973) also reported similar tooth lives]. The frequent replacement of teeth was an additional cost to the project as there is considerable time lost in such an operation in extremely cold temperatues.

In more readily excavated soils, such as silts or sands of low moisture content, it was necessary to make only one pass, parallel to the excavation length. In this case, the ripper would usually start at one side and progress with successive passes across the width of the excavation. It was common for the ripper to be extended the full depth into these materials and for the tractor to pull the ripper with little difficulty in second gear.

It was interesting to watch the operation of ripping pure ice. Surprisingly, pure ice was quite difficult to rip (see Photo 9). The primary reason seemed to be traction. The ripper tooth was able to make a 15 to 30-cm cut with each pass and the dozer traveled in second gear with no difficulty. However, if deeper penetration was attempted, the bulldozer would loose traction. It appeared that a greater effort was required to rip pure ice than a frozen soil because of lack of traction.

With regard to excavating materials in general, there is always the question of whether ripping is feasible. The Caterpillar Tractor

Company Performance Handbook (1977) attempts to answer this question by using seismic velocities to classify various earth materials as "rippable," "marginal," or "non-rippable." According to the production charts in this handbook, many materials are considered "rippable" by a D-9 tractor equipped with a multi- or single-shank ripper if the seismic velocities are below 2500 m/sec and "marginal" if between 2500 and 3000 m/sec. Mellor (1977) reports seismic velocities for in-situ frozen silt in the Fairbanks area to be in the range of 1900 to 2900 m/sec and for frozen gravels in the range of 3700 to 3350 m/sec. Then, according to the Caterpillar chart, only the weaker silts in the Fairbanks area are considered rippable and the stronger silts and all the gravels are "marginal" or "non-rippable." Therefore, the Caterpillar charts predict that the frozen silts and gravels in the Fairbanks area would be difficult or impossible to rip. However, the production records of the Chena Project show that ripping was difficult, but not impossible.

b. Excavation Below the Water Table.

Where excavation below the water table was required, lifts of 0.67 to 1 m thickness were removed. This procedure worked well in zones where the materials were inorganic silts and sandy silts. These materials allowed frost penetration to proceed below the water table fast enough to permit excavation to be performed in the frozen zones at all times.

The contractors used only their experience and judgment for determining how fast the ground would freeze ahead of the excavation and when equipment could move into the area for additional excavation. In general, it was felt that approximately 1 m of additional frost penetration was needed before beginning a new excavation. Air temperatures near -30°C are believed to be best for this type of excavation since the ground freezing time is short and it is not too cold for efficient operation of the equipment.

On this project, the excavations were rather long and narrow; thus, several pieces of equipment could work in one area by excavating to approximately 1 m depth, and then moving to another area where the frost penetration had progressed to a point where additional excavation could be accomplished. In this manner, a given number of pieces of equipment were kept busy by systematically excavating over a rather large area. The depth of 1 m is a somewhat arbitrary depth however, it is based on the fact that the rate of frost penetration is quite rapid near the surface and becomes progressively slower with depth. Therefore, to wait for frost penetration much beyond 1 m could seriously delay the progress of the work.

During construction, reasonably accurate control of the depth of excavation is important so as not to penetrate below the frost zone more than necessary. In actual practice, it is quite common for the ripper tooth to penetrate below the frost zone, thus allowing some water to seep into the area. However, the seepage area can easily be plugged

by the dozer working back over it, confining the water to a very small area and allowing it to freeze before the area is entirely flooded. When seepage appears in the trench, the ripper operator knows he is getting too close to the bottom of the frozen zone. If he continues to rip, and large blocks of materials are removed at the frozen/unfrozen interface (which are then removed by the scrapers) serious danger of flooding of the excavation can exist. Therefore, it is quite critical that the frozen blocks not be loosened down to the freeze/thaw interface.

Dense fog was often a serious problem in the poor light of winter. Whenever groundwater with a temperature of +2°C was allowed to seep into the excavation, a dense vapor cloud would be produced when the water encountered ambient air temperatures of -34 to -45°C. This cloud would remain in the area for two or three days, or until all free water was frozen. Photograph 8 indicates how the fog developed.

Considerable difficulty was encountered where exceptionally deep excavations were required and the soils had a relatively high organic content or were clean, highly pervious gravels. Because the rate of frost penetration in such materials is naturally much slower, the contractor experienced extreme difficulties in maintaining the continuously frozen zone beneath the excavation. This problem was compounded by the fact that the deeper he excavated below the water, the higher the potential head and, thus, it became more difficult to stop any flow of water which might break through the frozen curtain. On several occasions, the contractor did excavate through the frozen curtain while operating well below the water table and was flooded out by large quantities of water entering the excavation. In such cases, he was forced to abandon this procedure and go to the more conventional pumping methods utilizing a backhoe for the balance of the excavation work.

In the majority of these cases, since the contractor found that the pumping was very difficult in the severely cold temperatures, he abandoned all excavation in these areas until the warm summer season when pumping could be utilized. But this proved to be a very costly lesson for one contractor since the minor amount of material to be excavated which remained in these areas was very difficult to remove after the frost had disappeared from the ground. The area was too large to reach with a crane or backhoe from points on dry stable ground; therefore, he was forced to build ramps into the area to serve as a working surface for a dragline. This system also increased the amount of haul roads needed to remove these materials from the excavated area and made it difficult to inspect the foundation and ensure that all unsatisfactory materials had been removed.

FACTORS AFFECTING THE RIPPABILITY

For practical purposes, there are four major properties of a frozen soil that affect their strength and, hence, their ability to be disengaged by tractor-mounted rippers. These are: (1) type of soil

(i.e., clay, silt, sand, or gravel), (2) the mass density, (3) the gradation, (4) the temperature and the degree of saturation. At slow rates of applied stress, the strength of a given frozen soil is highly dependent upon the rate of loading (Vyalov, 1963) (Sayles, 1968). However, within the practical range of speeds (e.g., 4 km/hr) for tractor-mounted rippers, the rate of loading is fast enough to produce brittle failure in the frozen soil which is nearly independent of loading rate (Sayles and Epanchin, 1966) (Bailey, 1967) (Haynes et al., 1975). Consequently, at normal tractor speeds the strength or force required for ripping is nearly independent of the speed of the ripping.

The following discussion will address the influence of the four properties of the frozen soil:

a. Soil Types - Density and Gradation

The types of soils discussed in this report range from non plastic to slightly plastic soils through the entire grain sizes to gravels. The report also touches upon organic materials. Since plastic clays were not encountered in this project, nor were very coarse cobbly gravels or boulders, these soil types will not be addressed. In general, the fine-grained materials ripped more easily than the coarse-grained materials. However, the water content at which these materials exist in nature plays a very important role in their rippability. For example, experience has shown that saturated gravel is much harder to rip than a saturated silt at any given temperature. However, at the other extreme of moisture content, a gravel with a rather low moisture content would be very easily ripped, whereas a silt at the same moisture content could be harder to rip than the gravel. In the Fairbanks area, the soils are all somewhat stratified, and it is very common to note very small thin lenses of sand in large masses of silts. Such stratification was difficult to determine visually, but it was very obvious during the winter ripping operations as the large chunks or blocks would usually break apart along these stratifications. This breaking action along the sand seams can be explained by the well known phenomenon that moisture is drawn from coarse-grained soils by contiguous fine-grained soil layers and, therefore, the degree of saturation is considerably less in the sand seams. Thus, the weaker sand seams provide natural breaking planes for the mechanical rippers.

Within any type of homogeneous frozen soil, it has been found that the denser the soil is the more difficult it is to rip. Cutting tests performed in the laboratory on frozen Fairbanks silt (Bailey, 1967) agree with this field observation. The results of some of these cutting tests, presented in Figure 2, show that for frozen silt with the same degree of saturation, $\underline{S}$, the specific energy, $\underline{E}_s$, increases as the dry unit weight, γ_d, increases. The specific energy ($\underline{E}_s$) is the energy required to disengage a unit volume of soil.

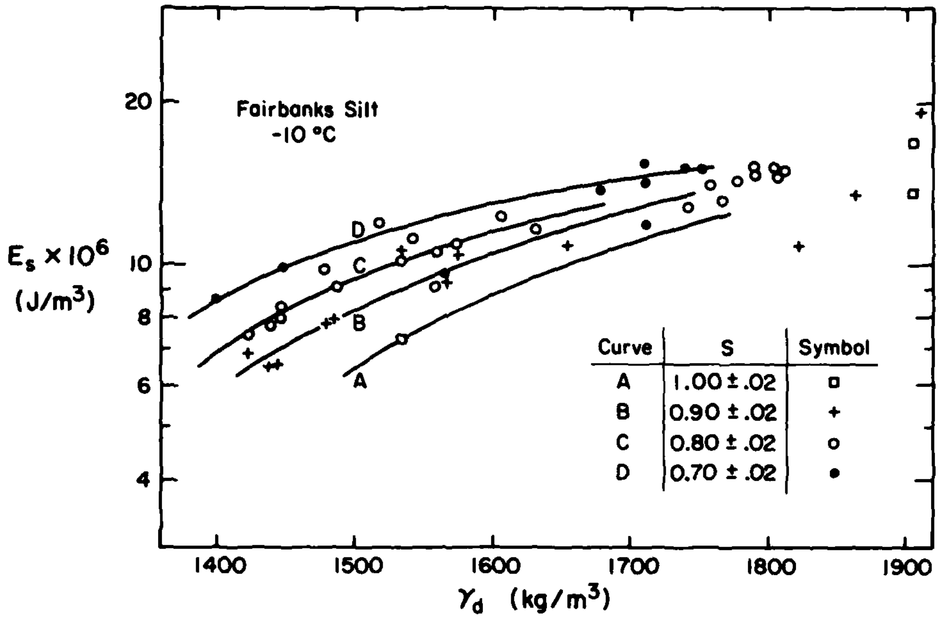

Figure 2. Specific energy, E_s, vs. unit dry weight, γ_d, for constant degrees of saturation, S [after Bailey (1962)].

The excavation of frozen peat was surprisingly difficult. Initially, attempts were made to remove the peat by scrapers until it was found that, in order to cut to a depth of only 5 cm, it was necessary to push a 24-m^3 double-engine scraper with two D-9 tractors. It was then decided to rip the peat before picking it up with scrapers. Even with ripping, the production was less than 150 m^3/hr per ripper. It was observed that, for a haul length of 610 m, three equipment hours of ripping were required for each hour of scraper time.

b. Temperature

There is no doubt that all saturated or partially saturated materials become increasingly harder to rip with decreasing temperature. This was very obvious in the Tanana Levee Project where extreme difficulty was experienced in ripping saturated gravels when the ambient air temperature was -40 to -46°C. At these temperatures, two D-9G tractors were required to obtain a few centimeters of penetration with a single ripper tooth. However, in April and May when the ambient air temperature was considerably warmer and ground temperatures were in the order of -4 to -1°C, a single tractor could rip the same material quite easily.

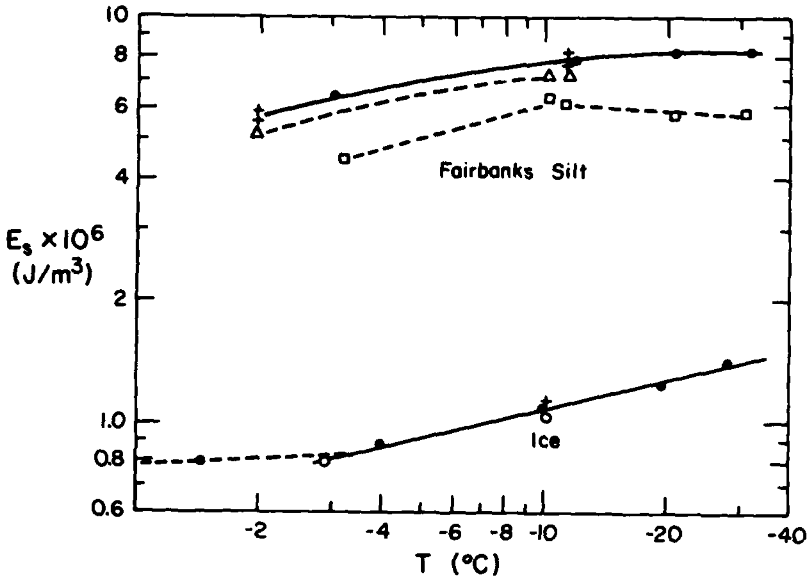

Figure 3. Specific energy, E_s, vs. temperature, T [After Bailey (1967)].

The results of laboratory cutting tests reported by Bailey (1967) on Fairbanks silt and ice also show that as the temperature decreases the energy required to disengage a unit volume of material E_s increases, as shown in Figure 3. It is interesting to note that for temperatures below -10°C the E_s is nearly constant for the silt while it continues to increase for the ice. Thus, it can be stated that the difficulty of ripping any material increases with a decrease in temperature. Frozen soils with very low moisture contents can be loosened at any temperature with about the same effort as that required to loosen unfrozen soils. Well drained sands or gravels that exist well above the water table or materials placed in stockpiles which are well drained often fall into this category.

c. Degree of Saturation and Water Table

As previously discussed, the moisture content of a soil has a direct bearing on its rippability. In general, it can be said that a soil becomes increasingly more difficult to rip with an increase in the degree of saturation up to where the voids in the soil are filled with ice. When excess ice is increased, the material becomes increasingly easier to rip, especially coarse-grained soils. One explanation for this decrease in strength is that the intergranular structure is being disrupted by the large ice content of the soil matrix; therefore, the strength of the soil mass becomes less dependent upon the intergranular structure and more dependent on the strength of the ice alone.

However, for frozen silt the relationship between energy for disengagement and degree of saturation is not completely clear. The laboratory data plotted in Figure 2 show that, at high values of dry unit weights, the effect of varying the degree of saturation is quite small within the range of 70 to 100 percent. However, at the lower dry unit weights the specific energy of removal clearly decreases with an increasing degree of saturation. These results are in apparent disagreement with qualitative field observations and with results of laboratory studies reported by Zelenin (1959). It appears that additional studies are required to clarify this relationship.

The relationship of a soil to the static water table is also an important factor in determining rippability. Obviously, this is directly related to the degree of saturation of the soil mass; therefore, the same relationships as stated above are true. Water table is important in the effect that it has on the degree of saturation or a particular soil mass. Coarse-grained, well drained materials which lie above the water table can be expected to rip quite easily in that the degree of saturation would be quite low. As we approach the static water table, the degree of saturation increases and the difficulty to rip also increases. In nonpermafrost areas, the static water table greatly influences the depth of frost penetration and, therefore, the rippability of any material. In the Fairbanks area, it is quite common to find seasonal frost penetration of an excess of 3.0 to 4.5 m in rather dry, well-drained materials; however, the seasonal frost seldom penetrates more than a meter below the static water tabel in similar areas. Therefore, the location of the static water table is an important parameter in determining the rippability of a particular soil mass.

PROGRESS OF EXCAVATION AND PROBLEMS ENCOUNTERED

a. Cold Weather Effects on Equipment and Personnel

When considering the feasibility, and production rates of excavation of frozen materials, one must take into consideration the effects on the equipment and the personnel. The efficiency of people and equipment working in the extreme cold and under conditions of poor lighting is considerably less than during the summer season. The following brief summary of the experiences reported by Williams and Costa (1976) on this project may be helpful in understanding some of the problems encountered. Although the contractors were able to move and operate equipment at extremely low temperatures, the usual policy was to stop all work when the temperatures were colder than -37°C. After periods of very low temperature lasting longer than 48 hours, there were problems of starting the equipment. Consequently, a special small crew of men usually reported 4 hours early to start the equipment for the day's operations.

The need for equipment maintenance increased during these extremely cold periods. There was an above 50% increase in hydraulic line and

pump failure during a startup in -35°C temperatures. About 35% of the diesel engines required major overhaul during these cold months. In most cases, the need for overhauling could be traced back to the practice of allowing the engines to idle for long periods of time at low temperatures rather than stopping them and restarting in the cold. Thus, for efficient operation, it was found necessary to provide shrouds for the engines during winter operations. Although it was expected that the cold temperatures would cause an increase in fatigue-cracking of metal parts, there was no notable increase in this type of failure. However, it was found that tires in extremely cold temperatures became brittle and were easily punctured by small sticks or sharp stones. This was not only costly in terms of purchase and repair of tires, but more so in the equipment downtime since changing a tire at -35°C became a major-time-consuming operation. Statistics for this project indicated that the availability of equipment was reduced 2% for every degree colder than -30°C. For example, if a company normally had 90% of its dozers available under normal conditions (i.e., above -30°C), it should expect about 75-76% to be available at -37°C.

b. Effects of cold on personnel.

Low temperatures had little effect on equipment operators while the equipment was operating in a normal manner. All trucks, scrapers and loaders were required to have adequate heaters and enclosed cabs. Dozers were equipped with canvas covers and fans blew air across the warm engines to the operators. However, adverse effects were observed among the supervisory personnel who did not have the benefit of the heat from operating equipment. For this reason, most supervision was done from pickup trucks with foremen darting in and out to give instructions and staying out for only 5 to 10 minutes at a time. Even when equipped with Army arctic clothing, a man could not work efficiently outside with notebook and camera for more than 1/2 hour at -40°C. The cold was particularly hard on surveyors whose efficiency dropped considerably because of these low temperatures.

c. Lack of Daylight

One of the most important factors affecting winter production in the subarctic is the lack of daylight. On 21 December, there is less than 2 hours of daylight in the Fairbanks area. Short days and intense cold combine to create safety problems, especially with portable light sets, engine starting, visibility in fog created by engine exhaust, and water vapor created during the excavation (see Photo 8). It is not only difficult for the contractor to perform under these exceedingly difficult conditions, but it is equally difficult for field inspectors to ensure that the proper excavation lines and grades are being met. The combination of a lack of daylight and dense fog also causes morale problems among the construction workers. Conscientious workers are not able to perform as efficiently as they would like and therefore tend to become sloppy and lackadaisical in their work habits. This puts an

increased burden on the field inspection personnel, thus making their jobs increasingly more difficult under extremely adverse conditions. These conditions create an inherent inefficiency in production in that all final grading and detailed inspections must be during the period of limited daylight hours. Therefore, it becomes necessary to schedule the daily operations so that the work requiring little or no inspection is done during the hours of darkness and the critical work is done during the daylight hours. Because this is not an efficient method of excavation, it increases the cost of construction.

d. Effects of Low Temperatures on Production.

To evaluate the effect of low temperatures on excavation production, it is necessary to compare quantities of soil excavated per unit effort in the frozen state with those for the same soil and conditions in the unfrozen state. But unfortunately this type of information is not available for this project. In fact, a literature search did not yield this type of data. However, to roughly describe the effectiveness and the relative productivity of the ripping and scraping operations Table I was prepared for the period 21 October to 31 December. In this table the effectiveness for ripping is defined as:

$$\text{Effectiveness} = \frac{\text{Volume of material disengaged}}{\text{Energy expended}}$$

Effectiveness is the reciprocal of the specific energy $\underline{D}_s$ given in Figures 2 and 3.

The records do not explain the low productivity for the period 1 December to 15 December. It is suspected that there may be an error in the way the volumes were recorded. The next to the last column of Table I shows the ratio of the production of the ripping operation to that of the scrapers; i.e., the rippers worked on the average of 1.7 hours for each hour of the scraper. This average is small compared with that for the typical operation where, for a hauling distance of 600 m, three hours of ripping were required for each hour of scraping. To consider this table another way, the ripping of the soils would not be required if the work was performed during the summer months, so all the ripping effort was the result of the low temperatures. Also, the lower productivity of scrapers can be attributed to the reduced efficiency of winter operations and darkness.

To put the values for effectiveness in perspective, Table II has been prepared from information available in the literature and the project described in this report. Although the effectiveness of ripping appears to be low for the project described in this report, the important details of the other projects are not available for comparison. In any case, the contractor found it economical to excavate the frozen soils in the winter based on the use of excavating equipment and the use of frozen soil as a stable working or hauling platform.

Table I. Production and effectiveness of frozen silt ripping and scraping operations.

Period	Temp.* (°C)	Production: Ripper (m^3/hr)	Production: Scraper† (m^3/hr)	Ratio	Effectiveness (m^3/Jx10^8)
21-31 Oct	-0.5	102	275	2.7	9.9
1-15 Nov	-0.1	152	241	1.6	15.0
16-30 Nov	-2.8	124	158	1.3	12.0
1-15 Dec	-3.3	10	9	0.9	0.96
16-31 Dec	-6.7	47	72	1.5	4.6
Average	-2.6	87	151	1.7	8.5

* Average ground temperatures taken just below the ground surface of a nearby meteorological station.

† Based on scraper hauling distance at 600 m.

Table II. Effectiveness of ripping.

Equipment: Tractor	Equipment: Ripper teeth	Material: Type	Material: Temp(°C)	Effectiveness m^3/Jx10^8	Remarks	References
D-8	1	Sandy gravel[1]	-1	129	Test run	Lange (1674)
D-9	1	Silty gravel[2]	-2	28	Test run	Lange (1964)
D-8	2	Clay	*	12	For 8 hours	Lange (1964)
D-9	1	Sandy clay[3]	*	9.4	For 8 hours	Haley (1959)
D-9-G	1	Silt	-2.6	8.5	Avg. for 3 mos.	Observation
D-9-G	1	Peat	-2.6	15.	For 8 hours	Observation

1 Saturated with some excess ice.

2 Had in-place unit weight of 2087 kg/m^3, water content of 6%, and contained less than 18% silt and fine sizes.

3 Frozen to a depth of 1.5 m

* No temperature given in references.

CONCLUSIONS

The qualitative observations on the ripping of frozen soils for the Chena River Lakes Project indicate the following:

> Saturated frozen gravels are more difficult to rip than saturated silts or peat, but frozen gravels with low water contents are easier to rip than the silts at the same degree of saturation.
>
> As the dry unit weight of a given soil increases, the effort required to rip it also increases.
>
> As the temperature decreases, greater effort is required to rip a given soil.
>
> The effort required to rip a given frozen soil increases as the degree of saturation increases until the voids of the soil are filled; however, after the voids of the soil are filled, an increase in excess ice reduces the effort required to excavate the soil.

There is no doubt that large quantities of frozen soils can be excavated successfully in the winter in the Subarctic at ambient air temperatures down to -40°C despite difficulties caused by starting engines, brittle failure of tires, additional maintenance of equipment, darkness, fog and personnel morale. Also, this type of cold weather excavation can be economical from the overall operation standpoint when year-round utilization of equipment and personnel are considered and when the accessibility to the site over swampy areas depends upon frozen ground to support the heavy construction equipment. To improve the efficiency of the excavation operations alone, however, additional detailed studies need to be conducted. Such studies must take into account the properties of the frozen soil as well as the failure mechanism in the soil that occurs in the field during the ripping procedure. Also, construction personnel must keep detailed records of not only the quantities of material removed from the project but, in addition, information on the soil, including the ice content, in situ density, soil temperature, gradation, and plastic indices. With these types of information, perhaps real progress can be made in improving cold weather excavation in the Far North.

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Photo 1. Moose Creek Dam, Channel Excavation Ripper unit attached to D-9-G Dozer. Temperature -35°C.

Photo 2. Moose Creek Dam, Channel Excavation D-9-G Dozer ripping frozen silts. Temperature -35°C.

Photo 3. Moose Creek Dam, Channel Excavation TS-24 twin-engine scraper loading frozen pieces of silt. Silt has been loosened by D-9 ripper.

Photo 4. D-9-G dozer ripping frozen gravels. Temperature -26 to -29°C.

Photo 5. Ripping pattern after one pass with D-9-G ripper. Penetration is 5 to 10 cm. Temperature -26 to -29°C.

Photo 6. Ripping pattern after one pass with D-9-G ripper. Penetration is 5 to 10 cm. Temperature -26 to -29°C.

Photo 7. Ripping frozen sands and gravels for the channel excavation. Note the cross-hatched pattern.

Photo 8. Scraper loading frozen gravel. Note: "Steam" results when unfrozen materials are exposed to the air. Temperature -26 to -29°C.

Photo 9. D-9-G ripping ice in river channel. Ice is 1.2 to 1.8 m thick. Ripping was slow and difficult as dozer could not get good traction. Temperature -26 to -29°C.

Photo 10. Winter equipment storage trench under construction.

USING EXPLOSIVES TO EXCAVATE FROZEN GROUND

by

Rupert G. Tart, Jr.[1] and Lewis L. Oriard[2]

INTRODUCTION

In the fall of 1977 Northwest Alaskan Pipeline Company (NWAP) conducted a series of trench blasting tests in the Fairbanks area. The purpose of the tests was to demonstrate that controlled blasting could be accomplished near an existing oil pipeline with no detrimental effects on that line. The tests were conducted at three sites believed representative of conditions that will be encountered along the Northwest gas line alignment, two rock sites and one permafrost site. This paper discusses the results of the tests conducted at the permafrost site and the conclusions drawn from them.

SITE EVALUATION

Maps of the Fairbanks area showing the location of the permafrost test site are presented in Figure 1. The terrain at the site is generally flat and covered with scrub spruce trees as shown in Figure 2. The site was selected because it was accessible and was representative of a typical permafrost zone through which the NWAP pipeline would have to pass.

The site had been investigated many years earlier during mining operations in the area. Using the old borings and the new surface probings the depth to permafrost was determined over the proposed test zone.

The thawed material was found to be one to two feet in thickness in the area selected for the test. To explore further the conditions at the site and to determine representative soil properties, several refrigerated borings were drilled in the area and laboratory tests were conducted on representative frozen samples. A typical boring log from this area is presented in Figure 3. Typical laboratory test results used in classifying the materials are given in Figure 4. From these it can be seen that the primary stratum underlying the site consists of frozen silt. This stratum appears to have remained permanently frozen, thus justifying the classification of permafrost.

FROZEN VS NON-FROZEN SITES

One of the major differences between permafrost formations and unfrozen formations which require blasting is that permafrost is generally much more continuous. That is, there are relatively few unhealed joints or fractures passing through a stratum of permafrost.

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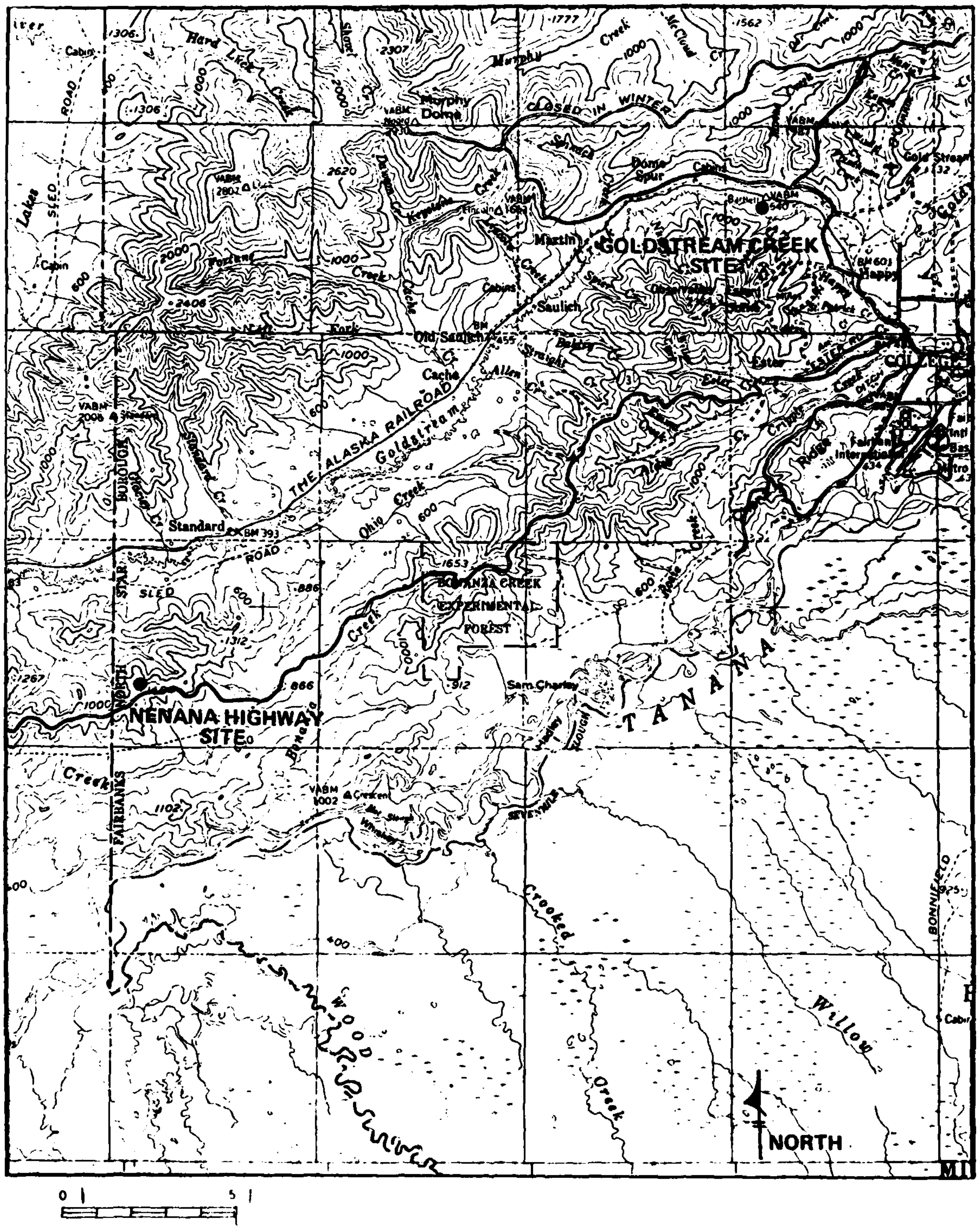


Figure 1. Test site locations north and west of Fairbanks.

Figure 2. Aerial view of Goldstream Creek site.

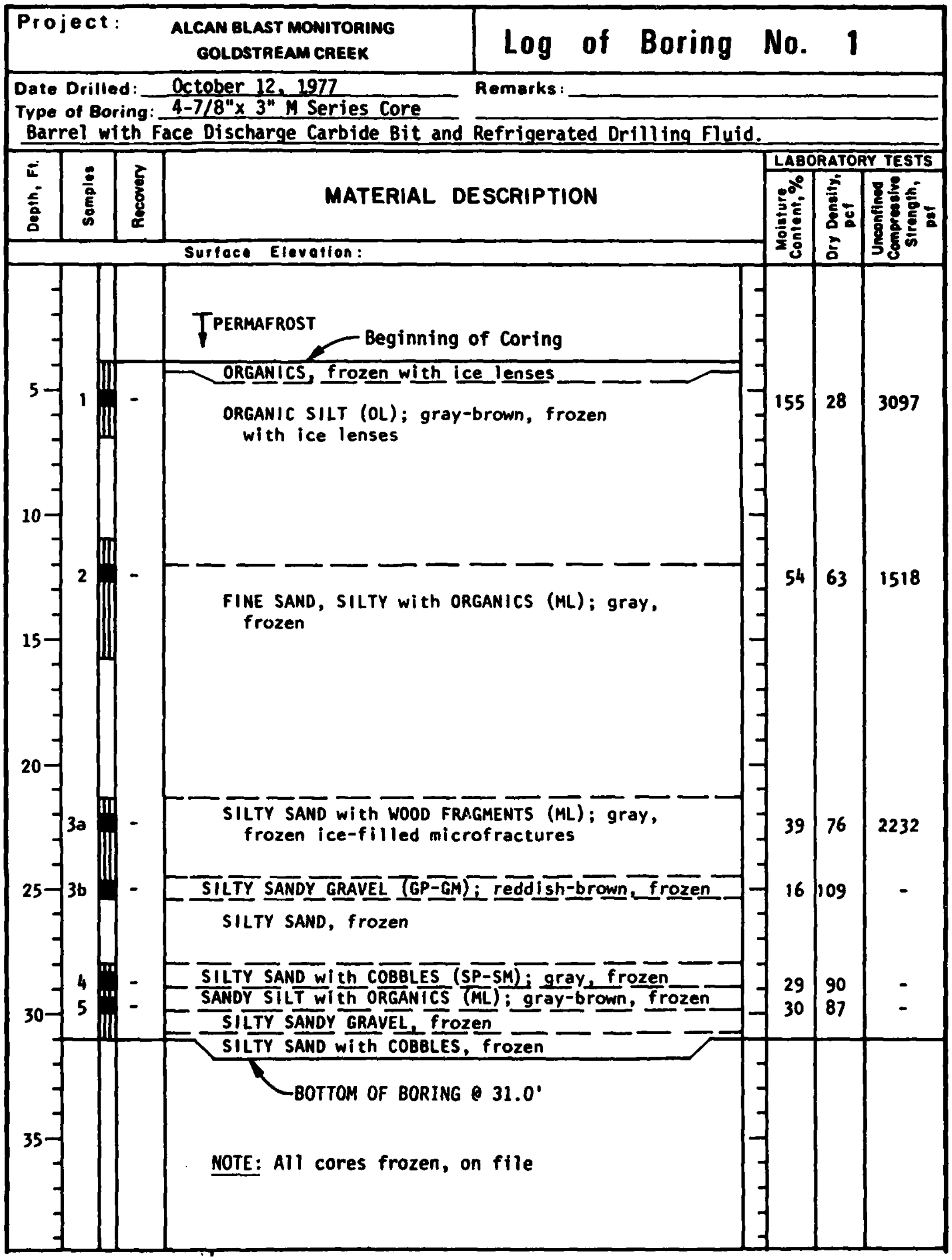
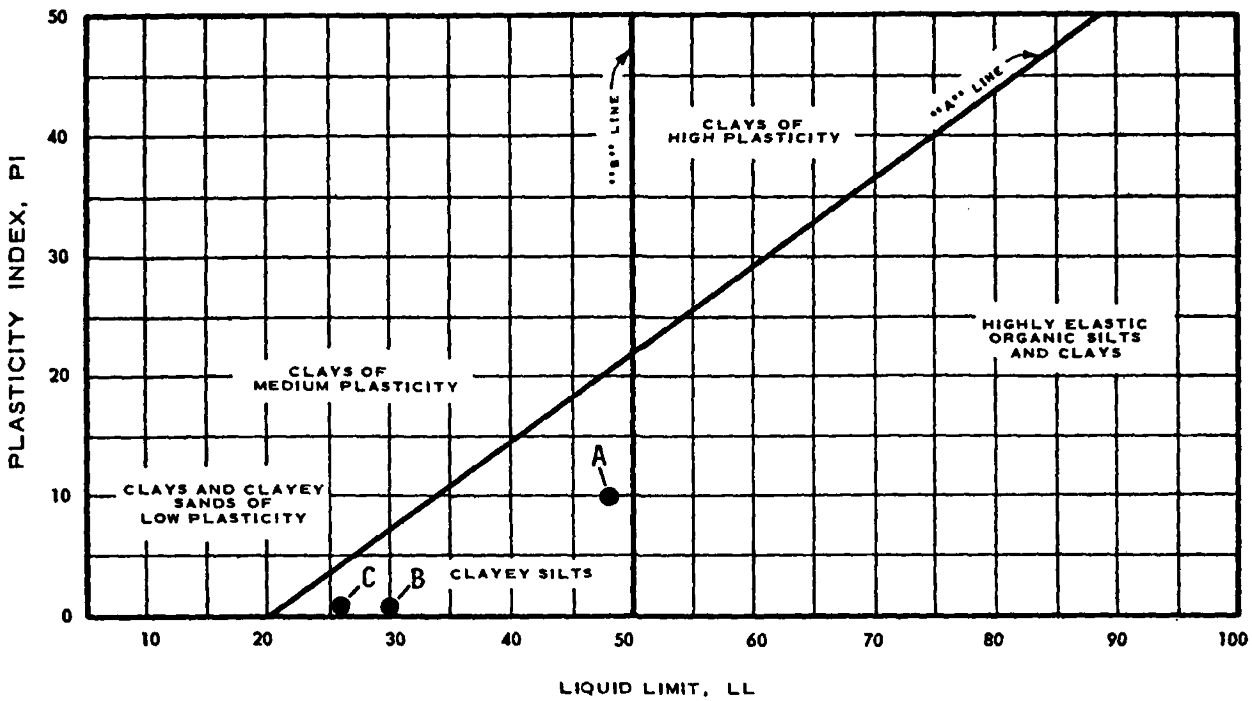


Figure 3. Log of boring no. 1.



CLASSIFICATION TEST RESULTS									
SAMPLE IDENTIFICATION			ATTERBERG LIMITS			GRAIN SIZES – % DRY WEIGHT			
LETTER DESIG'N	SAMPLE NO.	DEPTH, FT.	LIQUID LIMIT	PLASTICITY INDEX	PLASTIC LIMIT	SAND	SILT	CLAY	COLLOIDAL
A	1	5	48	10	38	4	—	—	—
B	2	12	30	1	29	9	—	—	—
C	3a	22	26	1	25	10	—	—	—

Figure 4. Plasticity classification, Goldstream Creek.

The prominent joints, fractures and other discontinuities that are found in most rock types are very important factors affecting the control of the geometric shape of a trench excavation. With the use of ordinary drilling and blasting methods, the pattern of these discontinuities will usually determine the shape of the trench, due to the displacement of rock blocks along the sides of the trench. The block displacement can also be a source of significant damage to adjacent structures or utilities.

In addition to the block displacement that occurs in the rupture zone adjacent to the explosive charges, there is often a significant venting of explosives gases along prominent weaknesses or open discontinuities in the rock mass. This venting can extend for surprisingly long distances in rock masses with open jointing, and is enhanced by the confinement that is typical of trench blasting. (Confinement inhibits the free motion of the rock in the desired direction, and permits more gas venting into the rock mass). Damage associated with gas venting can extend far beyond the normal dimensions of blast craters, or zones of rupture. (see Oriard, 1971).

Permafrost usually is free of fractures, joints, and other openings. When cracks form in permafrost, moisture tends to seep into the cracks and form ice which effectively heals the cracks. Thus, permafrost usually is more continuous and more elastic than jointed rock. By the same process, it becomes more uniform in its seismic behavior and hence more predictable in that sense. These features are not necessarily more favorable in long respects. For example, the increased elasticity means that seismic waves are attenuated more slowly than those passing through rock with closely-spaced open joints.

Regarding the limits of breakage, permafrost is relatively amorphous in its behavior, that is, it lacks a definite geometric pattern. Thus, it is more likely to break along a line of closely spaced blast holes than would be the case for jointed rock, but without such a line of blast holes to control the breakage, large slabs of permafrost might be shifted or ejected thus producing an unacceptable trench shape or a material which is difficult to excavate.

BLASTING TECHNIQUES

Various blasting techniques can be used to excavate trenches. For unrestricted blasting in open country, it is not uncommon to see the use of one or two rows of heavy charges to blast out a trench. With this technique, the trench cross-section is roughly V-shaped. Such a technique can produce a trench of adequate size if the charges are large enough and placed deeply enough. However, it would be unlikely that such a technique could be used to provide a trench of the uniform shape that might be required when the trench is in close proximity to other utilities. Therefore, controlled blasting techniques may be needed to limit the shape and dimensions of the trench, and to limit the ground vibrations.

In the controlled blasting tests for trench excavation near Fairbanks, small diameter linear charges were used to pre-split a fracture plane along the proposed trench perimeter. Additional small charges were placed in the rock mass within the trench. These small charges were used to break the material into acceptably small blocks for easy excavation. If the small charges within the mass are not properly distributed and delayed in firing sequence, it is likely that the permafrost would break into large slabs which would be difficult to excavate. This would also produce an irregular trench shape.

Figure 5 shows the actual layout of the charges used in these tests. The darkened circles on the left are the locations of the pre-split holes in which Primacord was placed to give a line charge in each of these locations. This loading scheme is shown in the middle section of the figure. These charges were set off instantaneously to pre-split, or pre-shear, the walls of the trench prior to the detonation of the production charges (shown as the open circles). The orientation of the production charges in the blast hole are shown to the far right and consist of two levels of charges in each of the holes. These were detonated in a delayed sequence in the numerical order shown above the open circles on the left. Each number represents a delay and there is a time lag of 25 to 100 milliseconds between each of the numbered delays. This delay aids in the breaking process and serves to reduce the vibrations by reducing the amount of explosive detonated at any one instant.

Figure 6 is a photograph of the two types of charges laid out on the ground. The cord on the left is 50 grain Primacord used to detonate the production charges. The cord on the right is 200 grain Primacord used for pre-splitting. The photograph also shows the sandbags used as stemming and the production charges. Figure 7 shows a typical trench blast just after detonation. In this particular blast, pre-splitting was done on only one side of the trench. The less-dense dust clouds to the right side of the blast are a result of the instantaneous pre-split detonation. On the left side of the trench, in front of the pre-split detonation, the more dense dust clouds were generated by progressive detonation of the production charges along the trench alignment.

One of the authors is currently engaged in research and construction usage of "fracture-control-blasting" techniques for the purpose of improving present pre-splitting methods. It is anticipated that perimeter hole spacings can be increased by a factor of about 3, and that charge concentrations can be reduced by a similar amount. With such improvements, these techniques can become a viable, economical approach to the control of trench excavation where blasting is required, even when the blasting is taking place in relatively restricted surroundings.

SEISMIC WAVES

Several types of seismic waves are generated by a blast in a solid material. There are two broad categories of these waves, namely body waves and surface waves. The first wave to arrive is a compressional

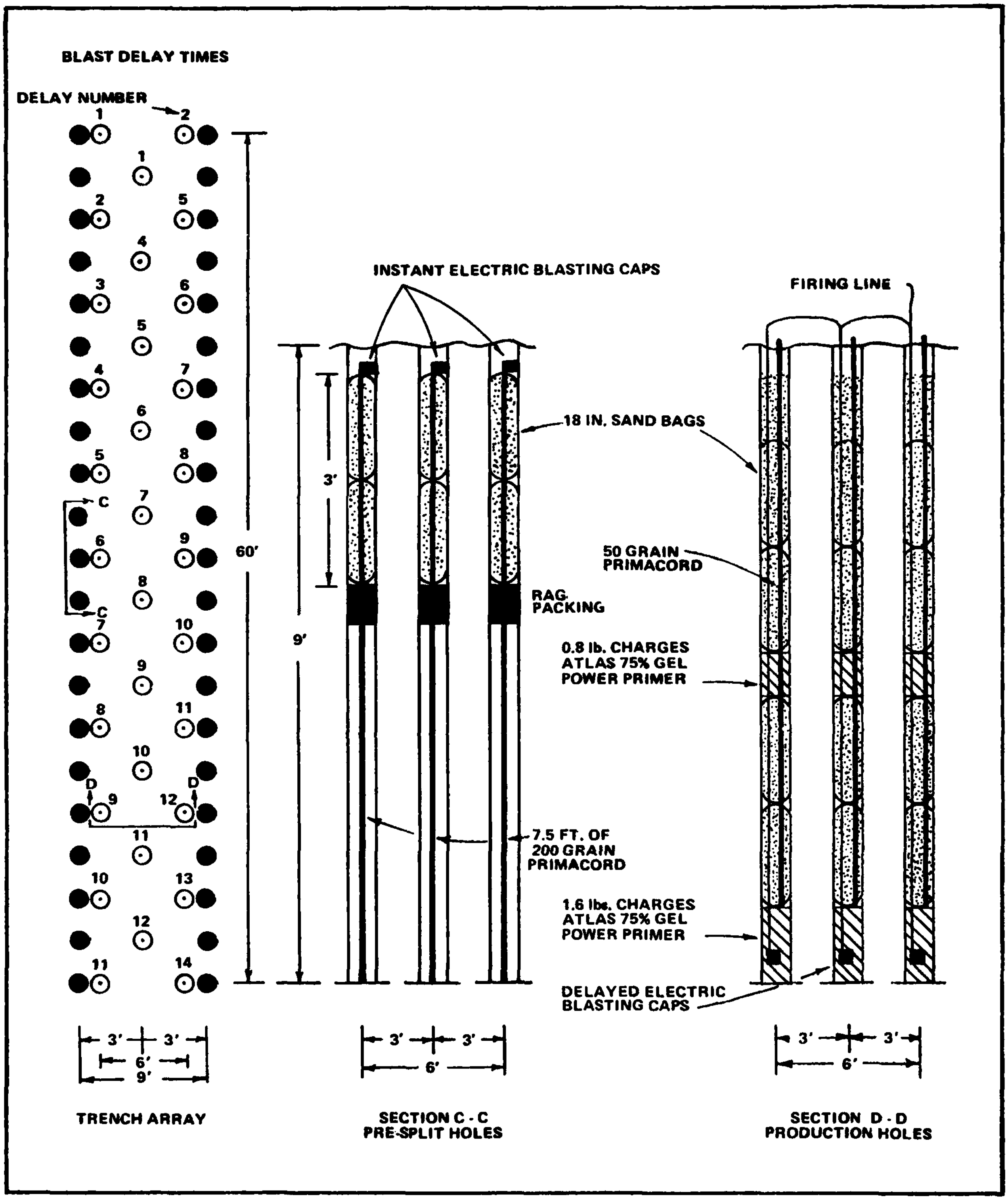


Figure 5. Operational test array.

SCALE

HOR. ½" = 10 FT
VERT. ½" = 1 FT

● PRE - SPLIT HOLE
⊙ PRODUCTION HOLE

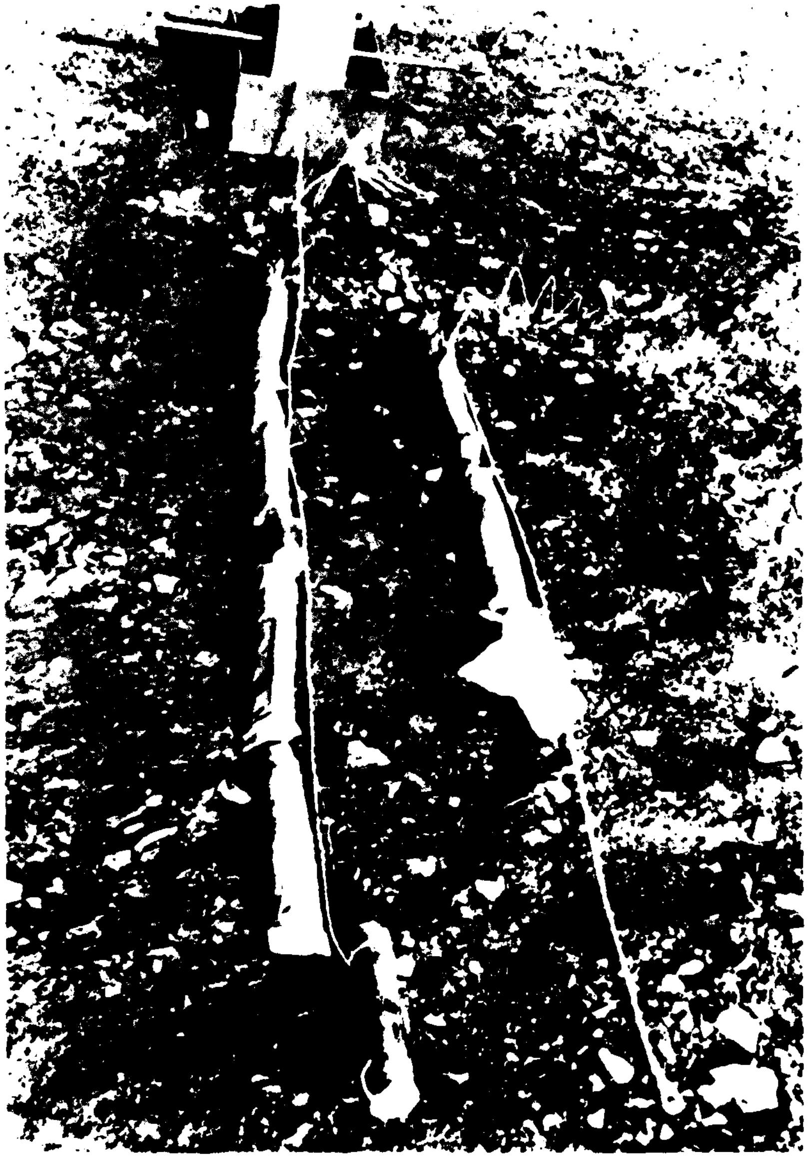


Figure 6. Typical production and pre-split loading schemes.

Figure 7. Trench blast showing pre-split and production.

wave. It is followed by a shear wave which usually has a larger particle displacement and a lower frequency. The surface wave of most significance is usually a Rayleigh wave, which is characterized by significant particle motion in a vertical plane parallel to the direction of wave propagation. Typically, it will have a larger particle displacement and lower frequency than any of the body waves. Because of the different propagation velocities of these waves, their arrivals become increasingly separated as the distance from the source is increased. In typical blasting operations beneath a relatively uniform ground surface, the surface waves predominate.

DAMAGE POTENTIAL

There have been various attempts to relate the different characteristics of particle motion to the potential for causing damage to structures. It is generally concluded that the peak particle velocity of the vibration can be correlated reasonably well to the damage potential, for vibrations

in the mid-frequency range which are common to most blasting operations (Oriard, 1970). The authors of this paper hold the opinion that all damage criteria are frequency dependent to some extent.

Figure 8 shows a simplified summary of criteria developed by Oriard and used herein as a guide. The criteria apply to several types of structures (see Oriard, 1978). In the opinion of the authors, it is unlikely that damage would occur to a typical residential structure if the particle velocity were less than about 4 inches per second. In some instances, residential structures have been known to tolerate particle velocities in the range of 10 to 25 inches per second while incurring either no damage or very minor damage. In turn, many engineered concrete structures have been known to tolerate vibrations with particle velocities in the range of 20 to 40 inches per second without suffering damage. Regarding steel structures, the authors know of no damage to steel structures from vibrations generated by nearby blasting. Well braced steel structures have been known to withstand particle velocities in excess of 200 inches per second (generated by strong earthquakes).

INSTRUMENTATION

Transducers responding to particle velocity were used in the tests near Fairbanks. These consist of small coils in an electromagnetic field which generate a small electric current in response to the ground motion. These can be oriented to pick up various components of particle velocity. In this particular test these transducers were placed in pipes to protect them from groundwater and were coupled to the bottoms of the holes (see Figure 9).

The transducers were mounted vertically to measure vertical velocity components and horizontally (see Fig. 10) to pick up the radial component of particle velocity. Transducer signals were fed to Sprengnether VS-1100 seismographs which recorded the motions during the blast. During the blast the seismographs were running continuously and produced records similar to that shown in Figure 11. The peak motion or maximum particle velocity is determined from this record by scaling. The transducers were located at four positions horizontally from each blast and at three different elevations at each of the four positions, as shown in Figure 12.

ANALYSIS

In order to correlate particle velocity with distance from the blast, it is customary to "scale" the distance by some power function of the charge weight per delay. The scaling factors most commonly used are the 1/3 power, the 1/2 power and the 2/3 power. Other characteristics of the blasting methods and geology can be included at the discretion of the investigator. The prediction curves used for this study are shown in Figure 13, after Oriard. These curves illustrate the use of square root scaling, and include factors for both geology and blast design. For further discussion, see Hendron and Oriard (1972), and Oriard (1978).

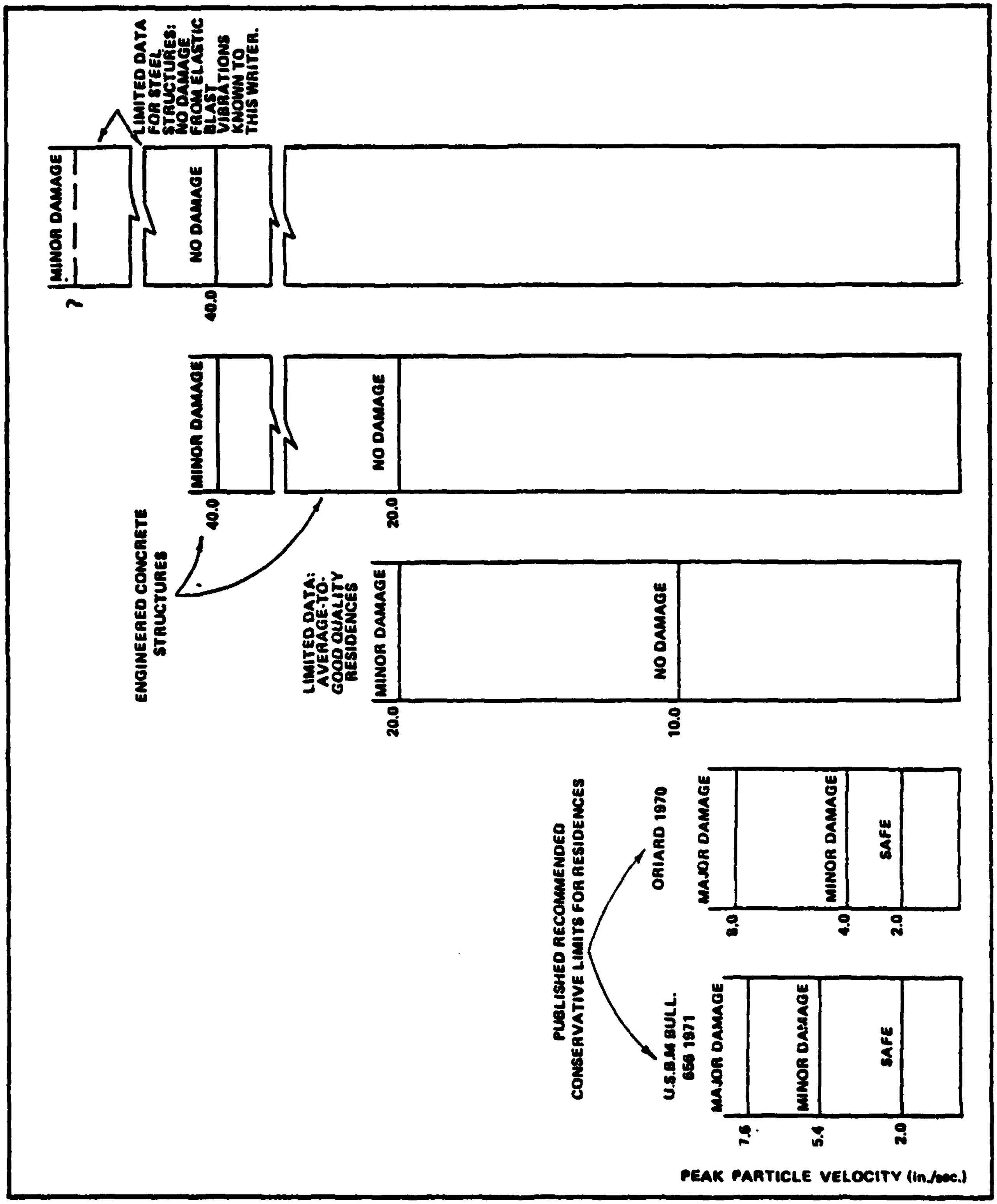


Figure 8. Tentative observational criteria for blasting vibrations in the mid-frequency range (limited data at high levels).

Figure 9. Placing pipes containing transducers.

Figure 14 shows that the data obtained during the tests conformed well to the predictions. It was of particular interest to confirm two of the predictions: 1) particle velocities tend to be higher in the more elastic, unjointed materials, and 2) particle velocities are higher at the ground surface than they are within the body of the material.

In addition to measuring particle velocities, we estimated the propagation velocities of compressional waves at the three sites (Table 1). It was observed that the Goldstream Creek site had a higher range of P-wave velocities than either of the two other sites, which were both underlain by highly jointed rock. Thus, the P-wave velocity data supported the basis for the particle velocity predictions.

Figure 15 graphically summarizes the results of the measurements at the Goldstream Creek site. This figure can be used for a refined prediction of the effects of this type of blasting in permafrost. For example, assume a distance of 30 feet and charge per delay of 9 pounds. The scaled distance would be 10. Projecting a scaled distance of 10 to the upper bound of the attenuation data gives a peak particle velocity of 2.3 inches per second expected at the ground surface. Of course, a different type of blasting would be expected to give different results.

Figure 10. Transducers mounted in pipe cap.

CONCLUSION

It has been proposed that the gas line to be constructed by Northwest Alaskan Pipeline Company will be located at a distance of approximately 50 to 80 feet from the existing Alyeska oil line when in the parallel configuration. The tests described in this paper illustrate that controlled blasting methods can accomplish the necessary trench excavation safely. These blasting techniques can be designed to limit the bounds of the trench excavation as well as to limit the ground vibrations generated by the trench blasting.

ACKNOWLEDGMENTS

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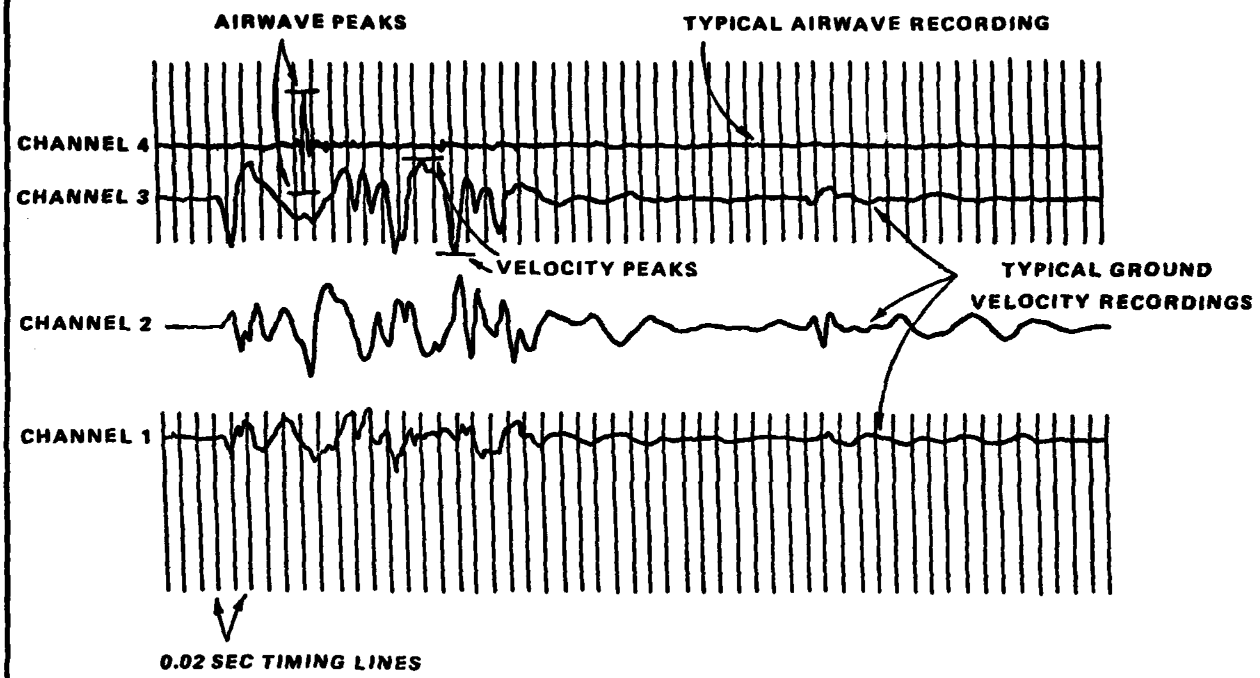

Figure 11. Typical recording from Sprengnether VS-1100 recorder.

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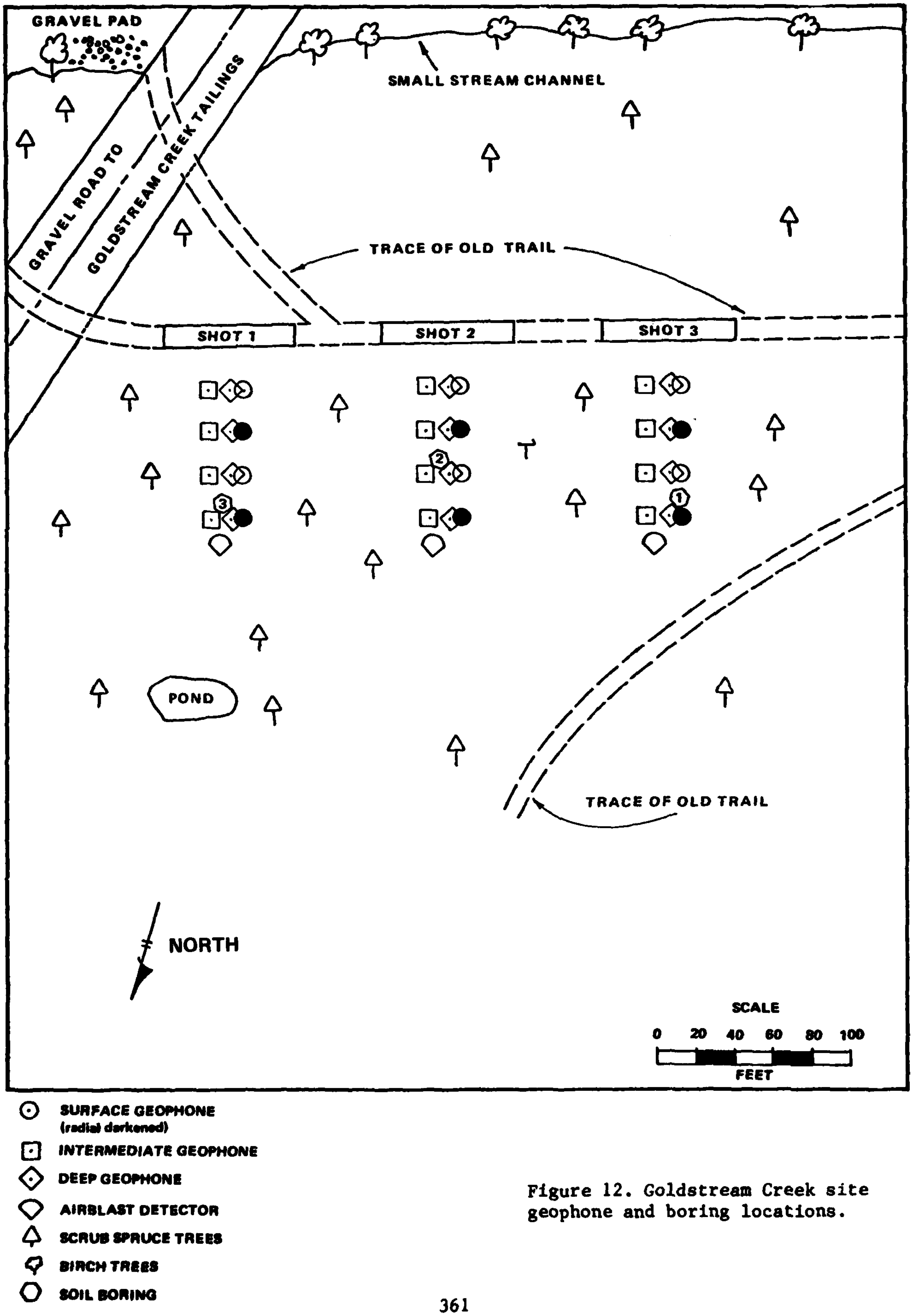

Figure 12. Goldstream Creek site geophone and boring locations.

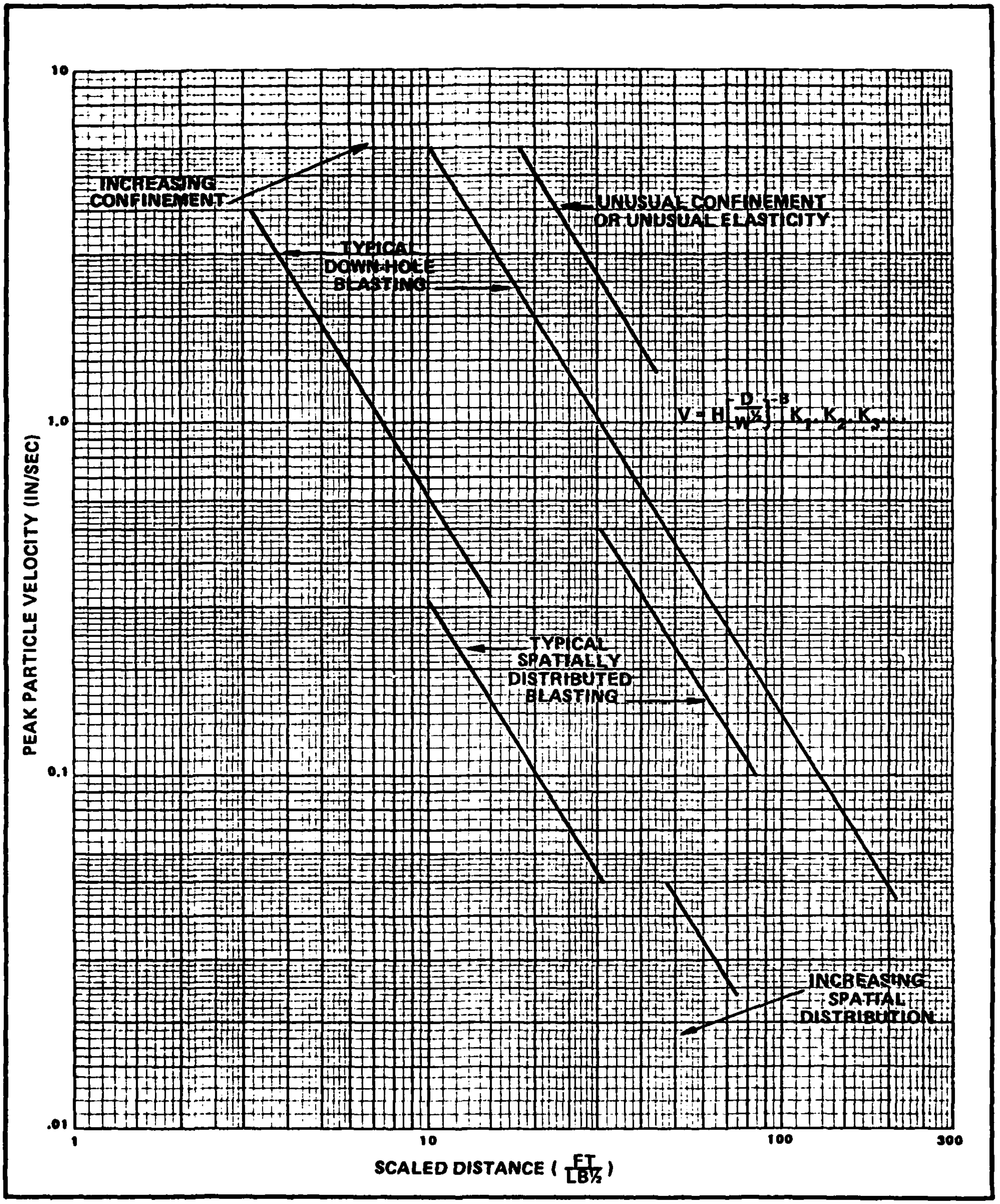

Figure 13. Oriard prediction curves.

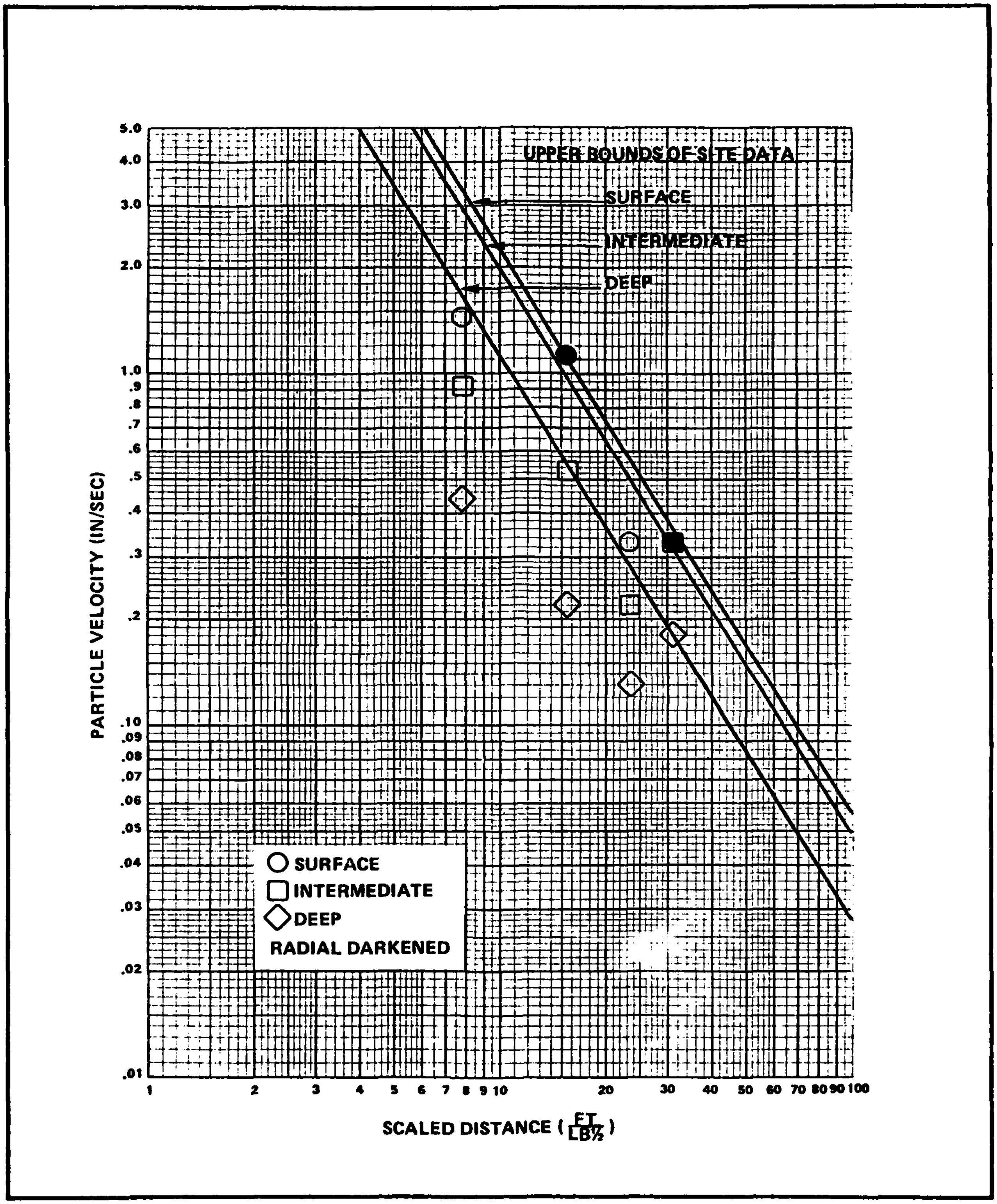

Figure 14. Plot of data particle velocity vs. scaled distance Goldstream Creek - shot 3.

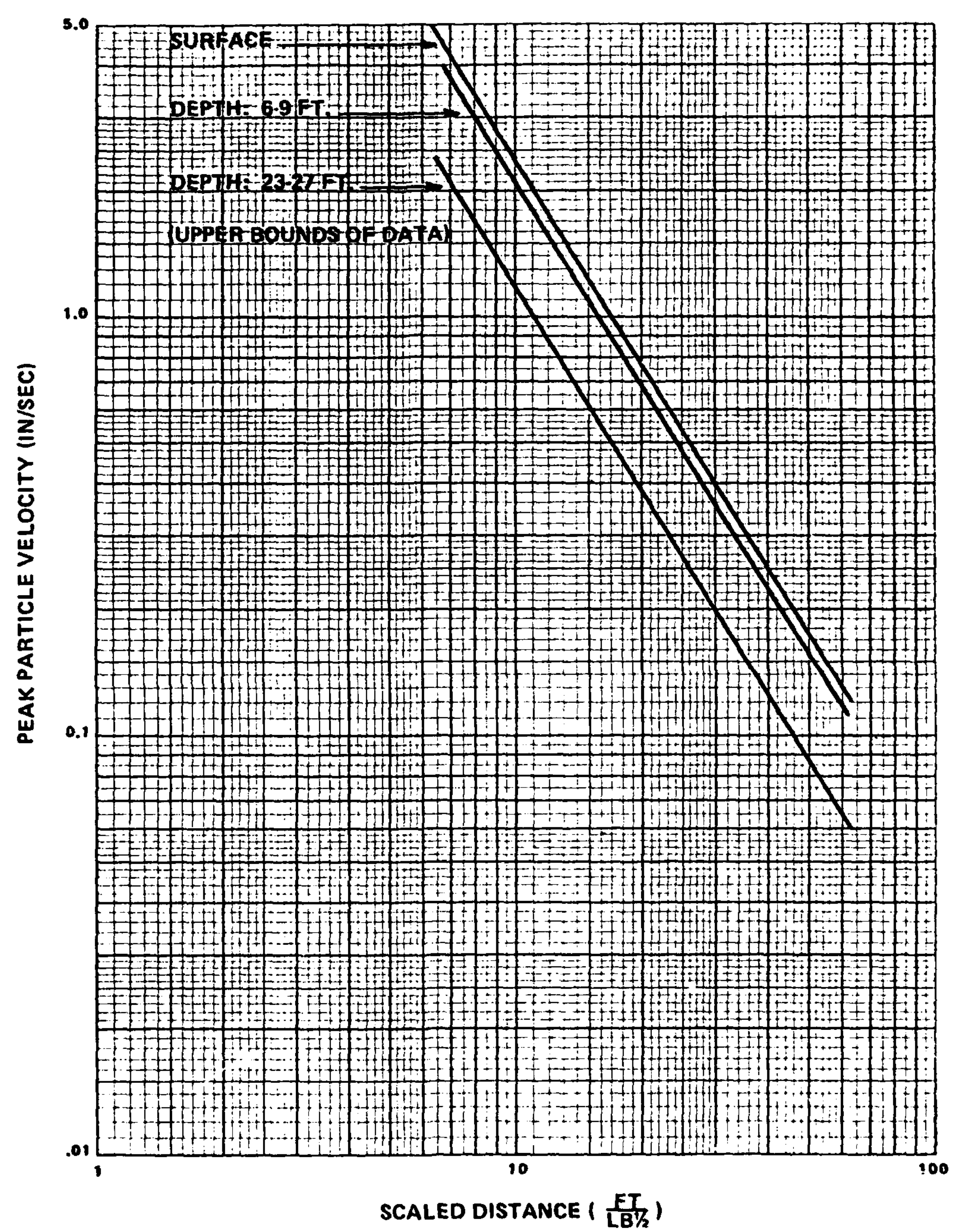

Figure 15. Summary of test results Goldstream site.

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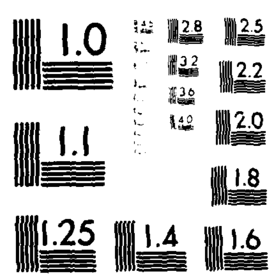

Table

Site
Nenana Highway
Goldstream Creek
Birch Lake

Table 1. Estimated Range of Seismic Velocities

Site	Range of seismic velocities (2000 ft/sec)	Estimated Rippability of site	Uniformity of site conditions
lenana Highway	1.4	Probably rippable	Low
oldstream Creek	9.11	Not rippable	High
irch Lake	7.10	Probably not rippable	Moderate

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